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Analysis of Reinforced Revetment Slope of Sargent Beach Erosion Protection Project on the Gulf Intracoastal Waterway

by Ronald E. Wahl, John F. Peters, Kris McNamara, WES
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Prepared for U.S. Army Engineer District, Galveston

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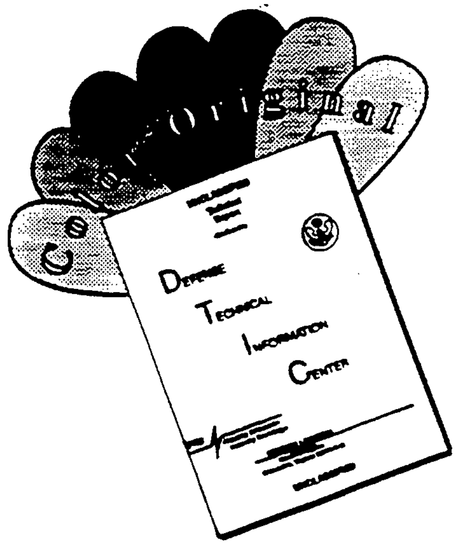

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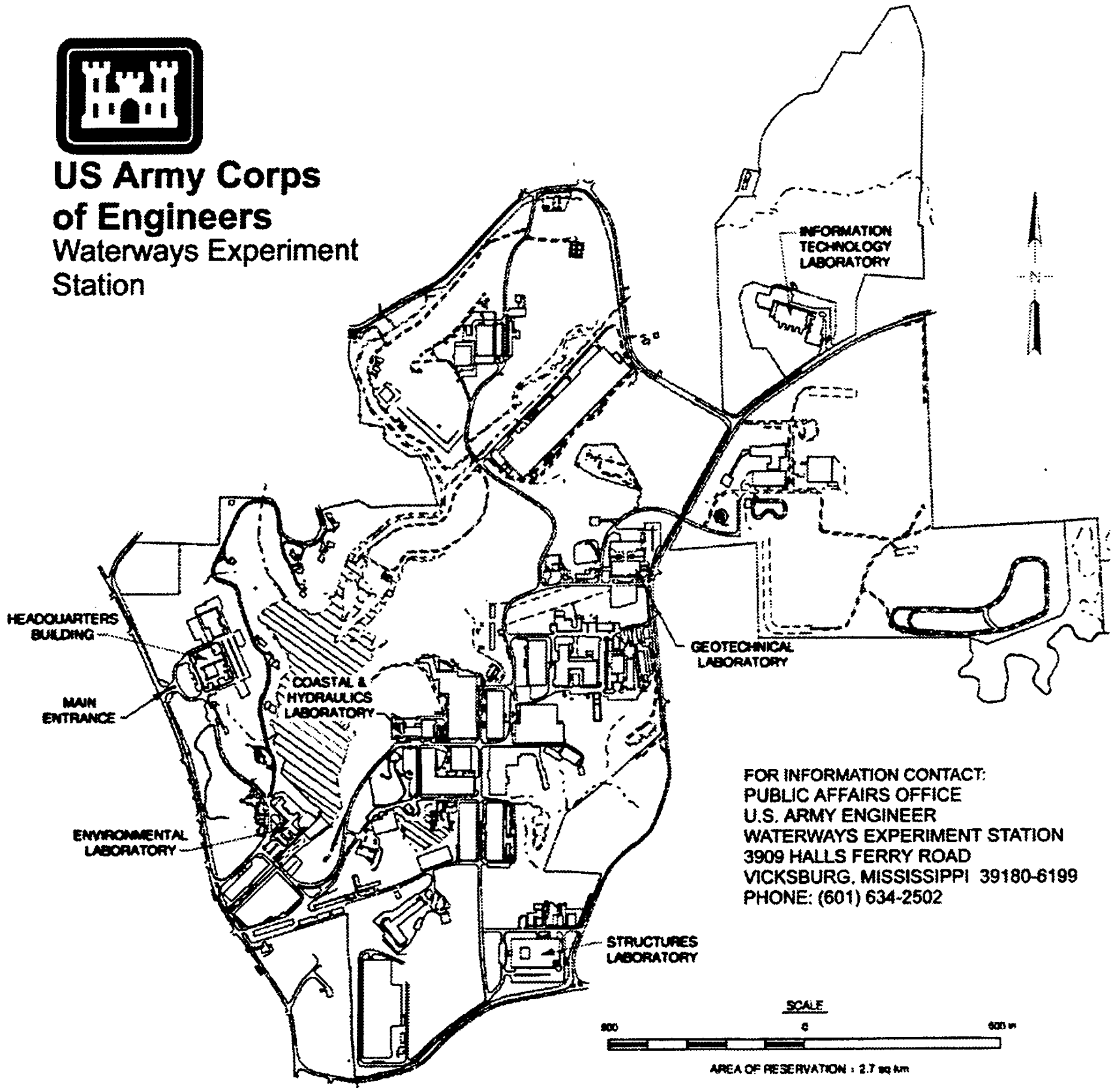

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Contents

Preface

This report describes the finite element and slope stability analyses used for the design of the reinforced revetment slope for the Sargent Beach Erosion Protection Project. Funding for this project was provided by the U.S. Army Engineer District, Galveston (CE-SWG), and the Directorate of Research and Development, Headquarters, U.S. Army Corps of Engineers, through the Civil Works Research and Development (CWR&D) Soils Program work unit entitled "Design of Earth Structures with Reinforcement." STUBBS, the computer program used for the finite element analysis, was developed by Dr. John F. Peters, U.S. Army Engineer Waterways Experiment Station (WES), Geotechnical Laboratory (GL), Soil Research Facility (CEWES-GS-GC), Soil and Rock Mechanics Division, under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program work unit entitled "Allowable Deformation of Earth Structures" and the CWR&D Soils work unit entitled "Large Deformation of Soils."

The analysis of the reinforced slope at the Sargent Beach Erosion Protection Project was performed at WES by Mr. Ronald E. Wahl, and Dr. John F. Peters, GS-GC. Ms. Kris McNamara, GS-GC, provided valuable assistance with the computer graphics employed for many of the figures used in this report. Mr. Ira Brotman, CE-SWG, provided input pertaining to the site characterization and assisted in the slope stability and finite element analyses. The work was accomplished under the general direction of Mr. David Bennett, Chief, GS-GC, and Dr. Don C. Banks, Chief, Soil and Rock Mechanics Division, and Dr. William F. Marcuson III, Chief, GL.

Dr. Robert W. Whalin was Director of WES and COL Bruce K. Howard, EN, was Commander at the time of the publication of this report.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	25.4	millimeters
kip-feet	1355.818	newton-meters
kips (force) per square foot	47.88026	kilopascals
kips (force) per square inch	6.894757	kilopascals
miles (U.S. nautical)	1.852	kilometer
pounds (force) per linear foot	14.5939	newtons per meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	0.006894757	megapascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter

1 Introduction

Background

The Gulf Intracoastal Waterway (GIWW) is located along the gulf coast of Texas and serves as a passageway for barge and ship traffic which carry goods and commodities to and from ports and harbors located along the east coast. The GIWW was designated as part of the national inland waterway system by the Revenue Act of 1978. As such, the U.S. Army Engineer Galveston District (CE-SWG) is responsible for the design and maintenance of the GIWW.

Erosion is a particular problem faced by the CE-SWG in maintaining the GIWW. Specifically, in a particular 8-mile segment near Sargent Beach the width of the land barrier has decreased to between 600 and 900-ft. The erosion rate in this segment has been observed to be between 25 and 36 ft per year. Thus, the CE-SWG is involved in the design of an erosion barrier to ensure that the GIWW will be protected from erosion. The location of the Sargent Beach Erosion Protection Project is shown in Figure 1.

The erosion control project will consist of the construction of a structural barrier which is approximately 8-miles (42,000 ft) long. The barrier will be placed 300-ft seaward and parallel to the GIWW. This width will save as much land as possible yet allow construction landward of the tidal range. The top elevation of the barrier was established at Elevation +7 to provide protection slightly above the top of the existing ground which is at approximate Elevation +5. The top elevation also corresponds closely with the surge elevation having a frequency of once every 10-years.

The type of structural barrier depends upon the subsurface conditions along the alinement of the GIWW. The current design plans allow for 36,600 ft of precast concrete revetment block and 5,440 ft of precast sheet-pile wall. The precast concrete revetment block will be along segments where the foundation conditions are strong enough to support the weight of the blocks. In these sections the block will be placed on 2-ft blanket stone (0.5 - 200 lb). The blanket stone will be placed on an excavated and prepared slope of 1V : 2.5H. The prestressed precast sheetpile wall is planned for use in stretches having weak foundation conditions. The design process required that piles must be 16-in. thick and 40-ft long to resist the maximum bending stresses. The piles will be composed of Type II or Type III

concrete. The reinforcement is to be ASTM A 615, Grade 60, and will be epoxy coated to resist corrosion. Pretension strands will be ASTM A 41, Grade 270k. Pile joints will be grouted to prevent the loss of fill through the joints.

In their cost analysis, CE-SWG determined that the construction of the sheetpile wall was an expensive design option for this project. Thus, they sought a less expensive alternative that would still meet the project requirements. The potential for significant savings exists if the sheetpile wall is replaced by precast revetment blocks placed on an excavated slope of 1V: 8H. A cross-sectional view of this design option is presented in Figure 2. The design calls for the placement of geosynthetic reinforcing material to provide additional stability to the slope.

Purpose

The CE-SWG tasked the Waterways Experiment Station (WES) to perform a finite element analysis to aid in the design of the reinforced slope faced with the concrete revetment blocks. The finite element method has the ability of simulating the construction process and predicting the behavior at different stages in the construction process.

In this study, the principal objective of the finite element analysis was to provide insight toward evaluating whether or not the predicted behavior of the slope would meet the project requirements. The direction and magnitude of foundation movements and forces in the reinforcement at different stages of construction were key items of information sought from the study. The effect of reinforcement stiffness on the slope's predicted performance was evaluated as part of this study to assist the designers in the selection of an appropriate reinforcing material. All analysis reported herein were performed under total stress and undrained conditions. These results were applicable to the short-term performance of the embankment.

Scope

The report is divided into five chapters including the introductory comments included in this part of the report. Chapter 2 contains presentations of the idealized soil profile and foundation conditions presumed to exist beneath the reinforced slope. The soil profile was drawn up by CE-SWG based on their subsurface investigation. Chapter 2 also contains a discussion of the sequence of construction activities simulated in the finite element analysis. Chapter 3 has a discussion of the analytical procedures used in this study. These include a limit equilibrium analysis using UTEXAS3 and the finite element analysis using STUBBS. The results of analysis involving the unreinforced cross-section and the effect of stiffness on the reinforced cross-section are also presented in Chapter 3. Some comments on the construction of the

project provided by the CE-SWG are included in Chapter 4. Finally, the conclusions drawn from this study are documented in Chapter 5.

2 Site Conditions and Construction Sequences

Idealized Zoil Profile

The idealized soil profile used in the analysis of the reinforced slope is presented in Figure 3. This profile was developed by CE-SWG based on the results of a subsurface investigation conducted during 1991-92 in the stretches where weak foundation soils occurred. All properties are consistent with those required for a total stress analysis representing the short-term or undrained conditions. The material properties needed for the analysis include the unit weight and undrained shear strength of each soil layer in the profile. Field vane shear and laboratory Q-tests were conducted to determine the undrained shear strength of each soil layer. The undrained shear strengths of each layer in the profile were conservatively selected based on an analysis of these data.

The idealized soil profile was made up of four layers. The top layer was described as consisting of a medium clay which extended from Elevation +5 ft to Elevation 0 ft. The second layer was a soft clay having an undrained shear strength of 290 psf and extending from Elevation 0 ft to Elevation -4 ft. The third layer was described as a very soft clay (probably normally consolidated) which had an undrained shear strength of 130 psf and extended from Elevation - 4 ft to Elevation - 24 ft. Subsequent analysis showed that this layer had a pronounced effect on the predicted performance of the reinforced slope because of its extremely low strength. This was underlain by a medium clay which had an undrained shear strength of 600 psf and extended from Elevation - 24 ft to unknown depth.

Construction Sequence

Details of the construction steps must be known to accurately model the performance of the reinforced revetment slope. CE-SWG has planned the construction to be carried out in six basic phases as shown in Figure 4.

PHASE 1. Figure 4 shows that the construction was initiated by making an excavation of 6-ft depth to Elevation - 1 ft. This excavation was made by removing the medium clay of Layer 1 and the upper foot of the soft clay of layer 2. This cut was necessary for the placement of the reinforcement. A sump pump was used to keep the excavation dry since the bottom is below the groundwater table.

PHASE 2. The reinforcement was placed at the base of the initial excavation (Elevation - 1 ft) and the initial excavation was backfilled with a compacted clay as shown in Figure 4. The compacted clay provided the ballast (normal load) necessary for developing the pullout resistance of the geosynthetic reinforcement. The reinforcement provided additional stability to the excavated slope as the construction progresses.

PHASE 3. The remainder of the required excavation was completed to Elevation - 9.5 ft as shown in Figure 4. The slope of the excavation is 1V:8H. The excavation removed the soft clays of Layer 2 and the very soft clay of Layer 3 down to Elevation - 9.5 ft. The cut will be flooded as the excavation progresses.

PHASE 4. Blanket stone will be placed along the slope of the excavation. The blanket stone will support the revetment blocks placed during Phase 5. The blanket stone were placed to a thickness of 24-in. along the surface of the slope and to a thickness of 18-in. above the compacted fill at the top of the excavation.

PHASE 5. The toe block and toe stone were placed at the bottom of the slope upon and adjacent to the blanket stone, respectively. The precast revetment blocks are then placed upon the blanket stone.

PHASE 6. Lastly, the construction was completed by placing the core stone and backfilling the excavation. This backfill will eventually erode away since it was placed on the Gulf side of the revetment stone.

Only Phases 1 through 5 were modeled in this study. The simulation of the construction of the reinforced revetment slope will be discussed in the next part of this report. The critical time for stability during construction was after the fifth phase because the excavation had attained its greatest depth and because the foundation soils were fully loaded by the surcharge placed on the slope by the weight of the blanket stone and revetment block. The long-term stability of the revetment slope was not evaluated as part of this study.

3 Analysis

The analysis of the reinforced slope was performed using both limit equilibrium and finite element methods. Total stress analyses were performed to evaluate the short-term stability of the slope. The cross-section of Figure 2 was analyzed both with and without reinforcement. Limit equilibrium calculations were performed with the slope stability program UTEXAS3 (Edris, and Wright 1993). The finite element analysis was performed with the program STUBBS (Peters, Wahl, Meade 1996). The finite element method offers the advantage of accounting for nonlinear soil behavior and the stiffness of the reinforcement and provides the designer with information about stresses and movements in the slope and foundations soils. However, the limit equilibrium offers the more conventional approach (Koerner 1991) and is useful because it provides a check on the finite element method, is relatively easy to use, and has features which allow its use in the design of reinforced slopes and embankments. Descriptions of the computer codes, computer models, analyses, and results are described in the following sections of this report.

UTEXAS3

UTEXAS3 is a general purpose program for evaluating the stability of embankments and slopes. Basically, the program uses Spencer's method to compute the factor of safety with respect to sliding for specified geometries and soil strengths. The program includes an option for including the effect of the force provided by a reinforcing element on the factor of safety.

In this study, UTEXAS3 was used to determine the safety factor of the unreinforced section and also determine the amount of force the reinforcement must supply to bring the system to equilibrium if the unreinforced safety factor is less than one. The cross-section and properties used in this analysis are shown in Figure 5. The effect of submergence was modeled with a piezometric line at Elevation + 1 ft and by the application of surface pressures on the slope at locations below Elevation + 1 ft.

STUBBS

The finite element analyses were performed using STUBBS (Peters, Wahl and Meade in preparation). STUBBS is a soil-structure interaction program which models nonlinear soil behavior using the endochronic constitutive model. The program has the ability to simulate typical construction processes such as fill placement and excavation by adding or removing elements from the finite element mesh during different time steps. Key items of information sought from the finite element studies include the stress, vertical and lateral displacements, and forces in the reinforcement.

The finite element method offers the important advantage of being able to account for the effect of the reinforcement's stiffness on the slope's performance. In this study, a range of reinforcement stiffnesses were investigated: (a) 90,000, (b) 200,000, (c) 500,000, and (d) 1,500,000 lb/ft. This range of stiffnesses was recommended for use in the analysis by CE-SWG as a representative range for the materials being considered for their design specifications.

Finite Element Model

The finite element model used in this study was designed to simulate the construction sequence for the reinforced slope which was described earlier. Overall, the mesh consisted of 706 soil elements, 12 reinforcement elements, and 2169 nodal points. Sixteen steps were used to simulate the first five phases of the construction process described previously. The steps used in the finite element study are outlined in Table 1. From the table it is apparent that all the construction steps involve either excavation and filling/placement operations. The finite element mesh will change at different times because these operations are performed by adding or deleting elements during a given constructions step. Figures 6 and 7 show the appearance of the finite element mesh at various times in the construction process. These are for step numbers 0, 3, 9, 14, 15, and 16. The zeroth step represents the initial conditions of a level ground surface.

Nine materials were used to characterize the solid elements in the cross-section. Up to seven parameters are required for each material. A description of each of the seven parameters is listed in Table 2. The parameter values used in the analysis for each of the eight materials are presented in Table 3. In this study, the strength and deformation properties were modeled in terms of total stress and no pore pressures were assumed in the analysis. All elements below Elevation +1 ft were assigned buoyant unit weights to simulate the effect of submergence in the analysis.

The reinforcing elements were treated as nonlinear elastic materials. These elements were formulated to have separate tensile and compressive stiffness. In this study, the reinforcements were allowed to take only tensile forces. Finite element runs were made with four different types of

reinforcements having stiffnesses of 90,000, 200,000, 500,000, and 1,500,000 lb/ft to evaluate the effect of different reinforcements on the performance of the slope.

Analysis of Unreinforced Section

UTEXAS3 analysis

A conventional slope stability analysis was performed to evaluate the safety factor of the unreinforced section, FS_u. Knowledge of FS_u is important in understanding the significance of the reinforcement in the stability of the slope. If FS_u is less than one than the slope's stability depends entirely on the ability of the reinforcement to carry any force deficiency necessary to keep the system in equilibrium. This force deficiency is caused by the inability of the foundation soils to carry the design loads. On the other hand, if FS_u is greater than one, then the reinforcement becomes a secondary defense against a slide and its purpose is to guard against uncertainty in the foundation strengths or other factors which may threaten the stability of the slope.

A model of the slope used in the UTEXAS3 analysis is shown in Figure 5. The analysis was performed using the UTEXAS3 search options for both circular and noncircular (wedge shaped failure surfaces. Safety factors were computed for three separate stages in the construction sequence. These stages include the time (a) when the excavation has been just completed to Elevation -9.5 ft (Step Number 14), (b) just after placement of the concrete revetment blocks at the end of construction (Step Number 15, and (c) just after placement of the concrete revetment blocks at the end of construction (Step Number 16). The results are shown in Figure 8. Significantly, this plot also shows that the critical wedges had lower safety factors than the critical circles for each of the three construction steps analyzed. *Thus, the wedge shaped failure surface is the critical failure mechanism for this problem.* For the wedges, Figure 8 shows that the safety factor for the unreinforced slope will decrease from about 1.52 to 1.33 as the slope is loaded with the blanket stone. The safety factor finally decreases to 0.923 after the revetment block is placed. *The analysis shows that the reinforcement is absolutely essential in providing stability for this combination of geometry and foundation strength since the critical wedge has a safety factor which is less than one.* Additionally, this lack of stability is controlled by the very soft clay layer (Layer 3) which had an undrained shear strength of 130 psf.

Finite element analysis

A finite element analysis was performed to evaluate the projected performance of the unreinforced section and to establish a link to the limit equilibrium analysis performed with UTEXAS3. This link is essential to tuning the interpretations of both the finite element and limit equilibrium calculations.

The results of the finite element analysis for the unreinforced case are presented in Figures 10, 11, and 12 which represent the results of construction step numbers 14, 15 and 16, respectively. Displacement vectors (showing the relative magnitude of movement and direction) for each nodal point are shown in each plot. The color in each figure shows the mobilized shear stress expressed as a percentage of the failure stress. In the figures, elements which are red have mobilized all of the available shear strength and have no reserve strength remaining.

Figure 10 shows that at Step Number 14 (after the excavation is completed to Elevation -9.5 ft) only a small portion of the elements have fully mobilized their available shear strength. Figure 11 shows that at Step Number 15 that the displacements increase (especially in the very soft clay layer) and that a greater number of elements have fully mobilized the available shear strength due to the increased load imposed by the stone blanket. The contours indicate that a failure surface is beginning to mobilize as indicated by the numerous elements colored red at the bottom of the very soft clay layer. Figure 12 shows that after completion of Step Number 16 the unreinforced slope has failed because a contiguous group of elements have mobilized all of their available shear strength. These elements are mainly located in the very soft weak clay which had an undrained shear strength of 130 psf. Additionally, the finite element solution did not converge on the sixteenth step which is another sign that the slope had failed.

The finite element results discussed in the previous paragraph are very similar to the results from the UTEXAS3 analysis. First, both methods predict that the slope will fail during the time when the revetment block is placed (Step Number 16). Secondly, the displacement vectors in Figure 12 show that the failure surface is a wedge located approximately in the same location as the critical wedge in the UTEXAS3 analysis (See Figure 8). These results show that the finite element solution is in good agreement with the UTEXAS3 results.

Analysis of Reinforced Slopes

UTEXAS3 analysis

The analysis of the unreinforced section revealed that the unreinforced safety factor (FS_u) of the slope at the end of construction was less than one (FS_u = 0.923). This result indicates that there is a force imbalance between the driving and resisting forces which will cause the slope to be unstable at this point in the construction process. The magnitude of this force deficiency will be the *minimum* force which the reinforcement must supply to maintain the slope's equilibrium. This force will be the value required to increase the safety factor of the slope to one.

This minimum reinforcement force was calculated with UTEXAS3 using the program's reinforcement options. Various values of reinforcement forces were specified as input to UTEXAS3. The forces in the reinforcement were

assumed to act in line with the orientation of the reinforcement. The critical wedge of Figure 9 was used as a failure surface and the safety factor was computed. The results of these calculations are given in Table 4 and are plotted in Figure 12. Figure 12 shows that a force of about 2700 lb/ft will be required to increase the safety factor from 0.923 to 1.0.

Finite element analysis

As mentioned previously, one of the principal objectives of the finite element method for this study was to evaluate the effect of stiffness on the expected performance of the slope. Four different stiffnesses having values of 90,000, 200,000, 500,000, and 1,500,000 lb/ft were used in the analysis. A separate finite element run was made for each case. Except for the presence of the reinforcement the finite element runs were made in an identical manner as that described earlier for the unreinforced slope. The following paragraphs describe the results of these runs. Key information sought in these runs included lateral displacements, vertical displacements, and the distribution of forces in the reinforcement.

Lateral displacement

Lateral displacement profiles from three separate locations were gathered from the finite element solutions. Figure 14 shows these locations were at sections 12 ft landward of the centerline (at the top of the slope at X = -12 ft), 8 ft to the gulf side of the centerline (at midslope at X = 38 ft) and 96 ft to the gulf side of the centerline (at the toe of the excavation at X = 96 ft).

The lateral displacement profiles for stiffnesses of 90,000, 200,000, 500,000, and 1,500,000 lb/ft are shown in Figures 15 through 18, respectively. In each figure, the top (X = -12 ft), midslope (X = 38 ft), and toe (X = 96 ft) profiles are presented in the left, center, and right plots, respectively. Each plot shows data for the fourteenth, fifteenth, and sixteenth construction steps. These plots show that generally lateral displacements increase markedly during the sixteenth construction step at which time the revetment blocks were placed on the slope.

Figures 19 and 20 contain plots which compare the effect of varying stiffnesses on the lateral displacements at the end of the sixteenth construction step. The displacement profiles for the top, midslope, and toe locations during the sixteenth construction step are presented in Figure 19. Plots of the peak lateral displacement in the midslope profile plotted against reinforcement stiffness are presented in Figure 20. The data in these two figures clearly show that the lateral displacements decrease as the stiffness of the reinforcement increases.

Vertical displacements

The vertical displacements of all nodal points located at elevation -9.5 ft were compiled from the finite element solution. This elevation is the level of the bottom of the excavation after the sixteenth construction step. Plots of the vertical settlement distribution at this level for reinforcement stiffnesses of 90,000, 200,000, 500,000, and 1,500,000 lb/ft are presented in Figures 21 though 24, respectively. Each plot shows the information obtained from data for the fourteenth, fifteenth, and sixteenth construction steps. As was the case for lateral displacements, the vertical displacements increase sharply during the sixteenth step when the revetment blocks are placed on the slope. In general, for each stiffness, the plots show that the maximum downward (negative) vertical displacement occurs near the project centerline (X = 0 ft). Heave (positive upward movement) begins near midslope (X = 38 ft) and increases to a maximum at the base of the excavation between X = 96 and 115 ft. Additionally, comparison of the data in Figures 21 through 24 after the sixteenth construction step shows that the vertical displacement decreases as the stiffness of the reinforcement increases.

Reinforcement forces

The force distributions in the reinforcements were calculated as part of the finite element analysis. This information is essential in the selection of the reinforcement. Figure 25 shows the force distribution at the end of the sixteenth construction step for the four stiffnesses studied as part of this project. This figure shows that the forces mobilized in the reinforcement depend on the stiffness of the reinforcement. The peak force is plotted versus stiffness in Figure 26. Both figures clearly show that the forces in the reinforcement increase as the stiffness of the reinforcement increases.

The peak forces computed in the finite element analysis are also listed in Table 5. The results indicate that as the stiffness decreases the peak force approaches the UTEXAS3 force required to improve the safety factor to one. This is because the stiffer reinforcements inhibit large movements in the foundation soils. The less the movement in the foundation soils the lower will be the percentage of the available shear strength mobilized. However, these smaller movements come at the expense of higher forces in the reinforcement because the stiffer reinforcement "attracts" load which would otherwise be carried by the shear resistance of the foundation soils. Conversely, since the FS_u is less than one, as the reinforcement becomes more extensible the full shear capacity of the foundation is approached and the force attracted by the geotextile will only be large enough to match the imbalance between the driving and resisting forces.

The concept discussed in the previous paragraph is illustrated in Figures 27 and 28. The plots on these figures show the percentage of available shear strength expressed as color contours. These figures show that a contiguous zone of fully mobilized shear stresses (red areas) is manifested only for the most extensible case where the stiffness equals 90,000 lb/ft.

Analysis of pullout resistance

Reinforcement pullout is a potential mechanism of failure which was not directly accounted for in the finite element analysis. In the finite element calculations, the geotextile was assumed to be compatible with the soil as no slipping between the two was permitted. In proper design it is necessary to insure that the embedment length is sufficient to develop the required tensile forces needed to stabilize the slope.

Gilbert, Oldham, and Coffing (1992) performed a laboratory investigation of the pullout resistance of geotextiles in *cohesive* soils. They investigated the effects of water content, the compaction process, normal pressure, loading rate and submergence on the pullout resistance between the geotextile and clay. In partially, saturated cohesive soils, they indicated that only about two-thirds of the soil's frictional component of strength can be relied upon at the soil-geotextile interface. In situations where the soil-geotextile is saturated (all pore space occupied by water and where the loading rate is rapid enough to prevent the pore water dissipation, the frictional component in the soil disappears and the lower limit of pullout resistance is the cohesion of the clay or the adhesion between the clay and the geotextile.

For the Sargent Beach Project, the geotextile will be sandwiched between the soft clay of Layer 1 and the cohesive compacted fill. These geotextile interfaces were submerged as the geotextile was placed at Elevation -1.0 ft which is below the elevation of the ground water table (Elevation +1.0). Thus, conservatively, the cohesion of the clay is assumed to represent the pullout resistance. Unfortunately, at this time there is no data on the pullout resistance at the geotextile-fill and geotextile-soft clay interfaces. However, a comparison of the pullout envelope with the force distributions for the various stiffnesses in Figure 29 indicates that the geotextile should not pull out provided the cohesion of the fill and soft clay layer are at least 150 psf.

Reinforcement performance

When assessing performance of the reinforced sections, it is important to consider the improvement in stability provided by the reinforcement. Referring to Figure 20, for reinforcement stiffnesses less than 10^6 lb/ft, the computed displacements begin to *increase rapidly* implying that the displacements have become more sensitive to stiffness for the more extensible geotextiles. This observation may be interpreted as follows: as the factor of safety in the soil is reduced to a critical level the displacements become less certain. Thus, there is an advantage to the higher stiffness beyond simply reducing the displacements; the greater the fabric stiffness, the less the soil strength is mobilized and the greater the reliability of the design.

4 District Comments

The Sargent Beach project was under construction at the time of publication. The contract was awarded to Luhr Brothers, Inc. for $42,648,694 in April 1995. The final design included 3,008 ft of 1V:5H sloped geotextile reinforced revetment, 4,468 ft of sheet pile wall, and 34,524 ft of 1V:2.5H sloped revetment. The contractor elected to construct the project using quarried granite block, trucked in from central Texas, instead of precast concrete block. An aerial view of the of a portion of the project is shown in Figure 30. A view of the in-place blanket stone and granite blocks is shown in Figure 31.

The finite element method gave the District an important option, by showing that a more gently sloped reinforced revetment, could be used in lieu of a sheetpile wall, in areas containing the poorest foundation conditions along the 8-mile reach. Refined cost estimates and constructability reviews concluded that a sheetpile wall would be easier, and as economical, to construct, as the revetment with a 1V:8H slope.

Areas adjacent to those displaying the weakest foundation conditions were reanalyzed with the appropriate design parameters. A limit equilibrium analysis (using UTEXAS3) revealed that the 1V:8H slope could be steepened to 1V:5H in those areas, better optimizing the design. One conclusion of the report was that UTEXAS3 and STUBBS agreed in their prediction of failure, and the force required by the geotextile to stabilize the slope. The information from STUBBS was critical in determining the stiffness of reinforcement specified to minimize the movement of the stone slope during construction, since this information could not be determined from a limit equilibrium analysis. Based on the results of STUBBS, the specifications require the woven geotextile to have an ultimate wide width tensile strength of 2,600 lb/in. (ASTI D 4595, and at 5 percent strain, a minimum tensile strength of 2,100 lb/in. (stiffness requirement).

Although the exact configuration of what was analyzed was not specified, the finite element method (STUBBS) provided much needed insight into how the slope would behave through the critical construction period. The estimated savings by using the reinforced slope, in place of the sheetpile wall, is $455,000. It is estimated that construction of the geotextile reinforced revetment will begin in the Spring of 1997.

5 Conclusions

The following conclusions were drawn from the limit equilibrium (UTEXAS3) and finite element (STUBBS) analyses performed to evaluate the stability of the reinforced revetment slope of the Sargent Beach Project on the Gulf Intracoastal Waterway. This analysis represents a worst case scenario due to the conservatism used in the selection of the undrained shear strengths of the natural soils occurring in the foundation.

a. The UTEXAS3 analysis showed that the unreinforced safety factor was 0.923. The critical failure surface was a noncircular wedge. This finding means that the geotextile reinforcement will be the primary line of defense in stabilizing the slope for the conditions of the analysis.

b. Both the UTEXAS3 and STUBBS analyses of the unreinforced slope indicated that the critical construction step will be during the placement of the revetment block.

c. The UTEXAS3 analysis revealed that the reinforcement must be able to supply a minimum of 2700 lb/ft to stabilize the revetment slope. This is the force required to improve the safety factor to unity.

d. For reinforced slopes, the STUBBS analyses showed that movements in the foundation will decrease as the stiffness of the reinforcement increases. The greatest increment in movement will occur during the construction step when the revetment blocks are placed upon the blanket stone. These movements are assumed to be those which occur immediately after construction of the slope and do not include consolidation.

e. The forces attracted by the geotextile increases as the geotextile's stiffness increases. This force increase represents the reduced strength mobilization in the soil that is responsible for reduced movements.

f. Pullout of the reinforcement should not occur provided the cohesive component of the cohesive fill and the soft clay between which the geotextile will be sandwiched are at least 150 psf.

g. Fabric selection and construction procedures should be directed toward achieving the greatest possible effective stiffness.

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Edris, E. V. and Wright, S. G. (1992). "User's Guide: UTEXAS3 slope-stability package, Volume IV: User's Manual," Instruction Report GL-87-1, USAE Waterways Experiment Station, Vicksburg, MS.

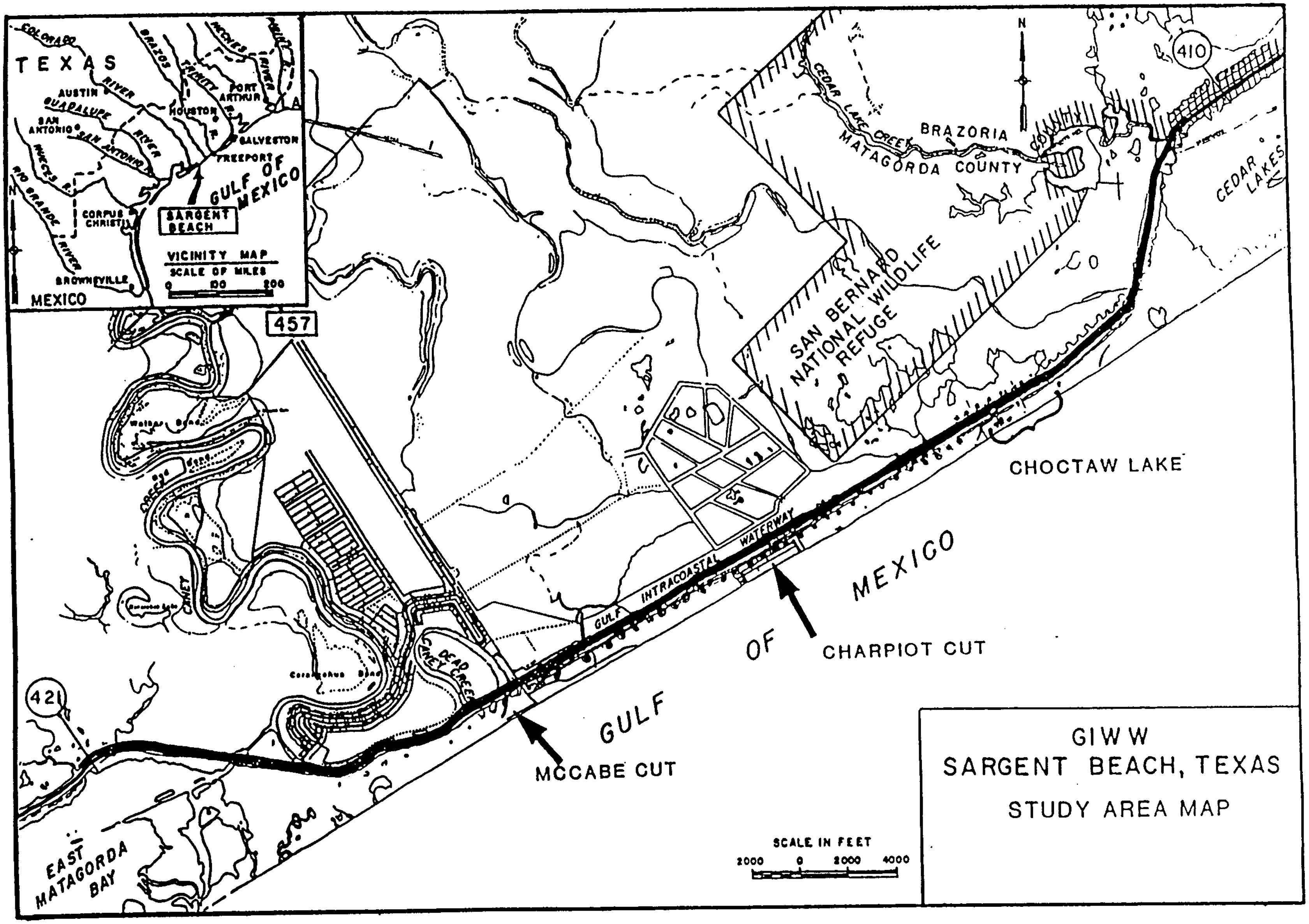


Figure 1. Locality sketch

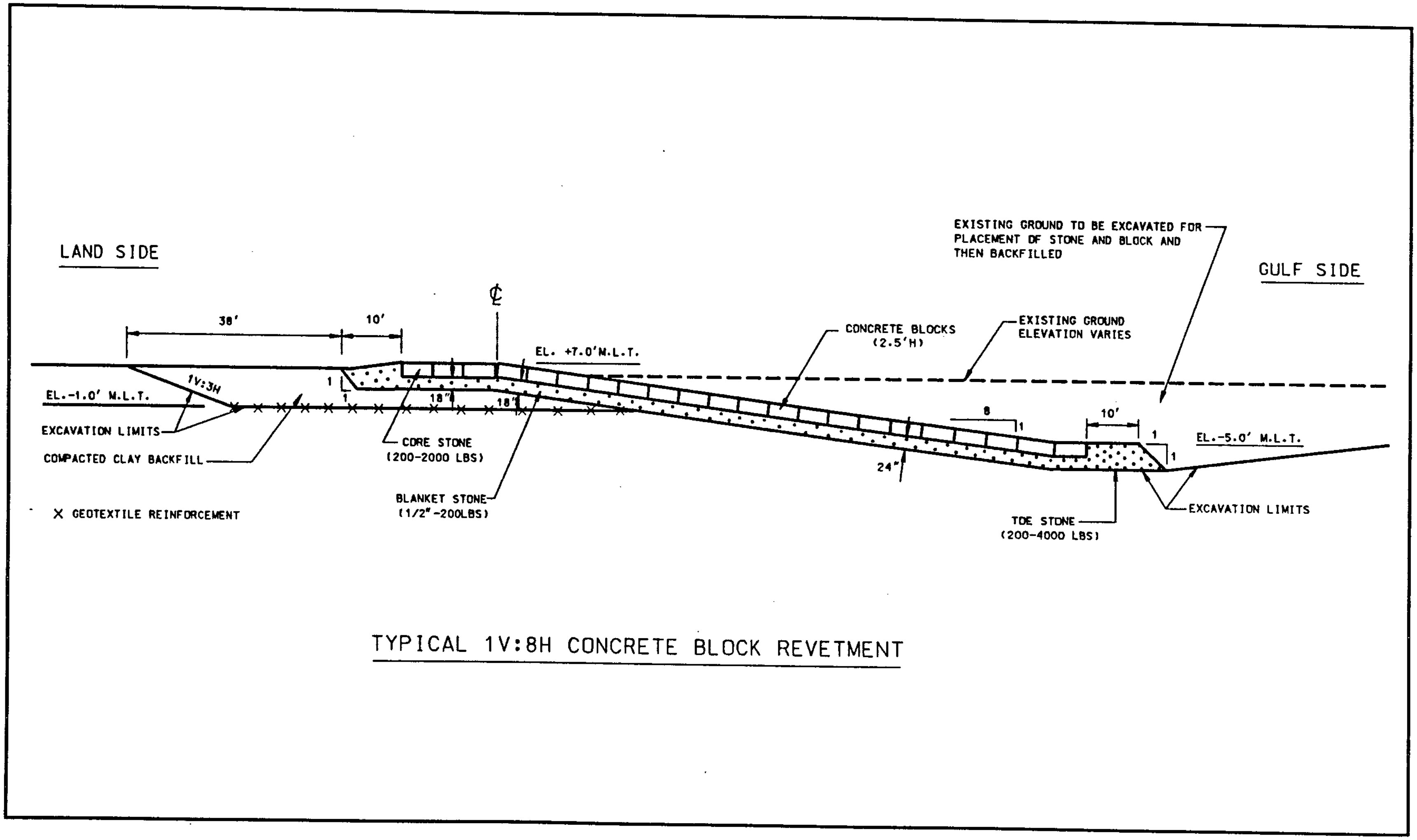
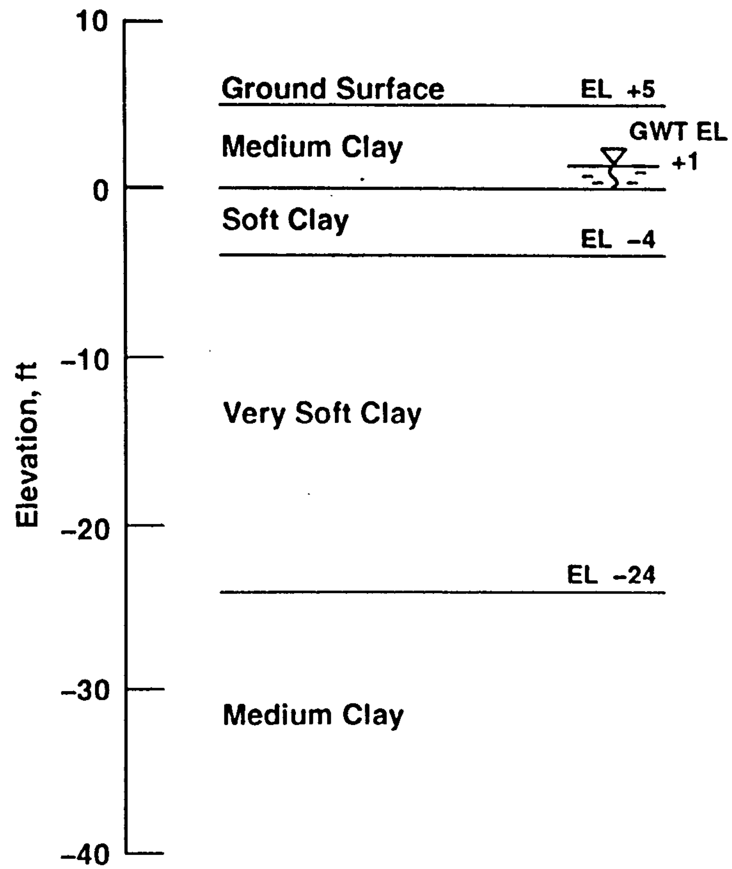


Figure 2. Cross-section of revetment with 1H:8V reinforced slope



Properties

Layer No.	γ_m pcf	S_u pcf	LL	PI
1	119	600	50	30
2	119	290	38	23
3	113	130	43	26
4	108	600	65	40

Figure 3. Idealized soil profile

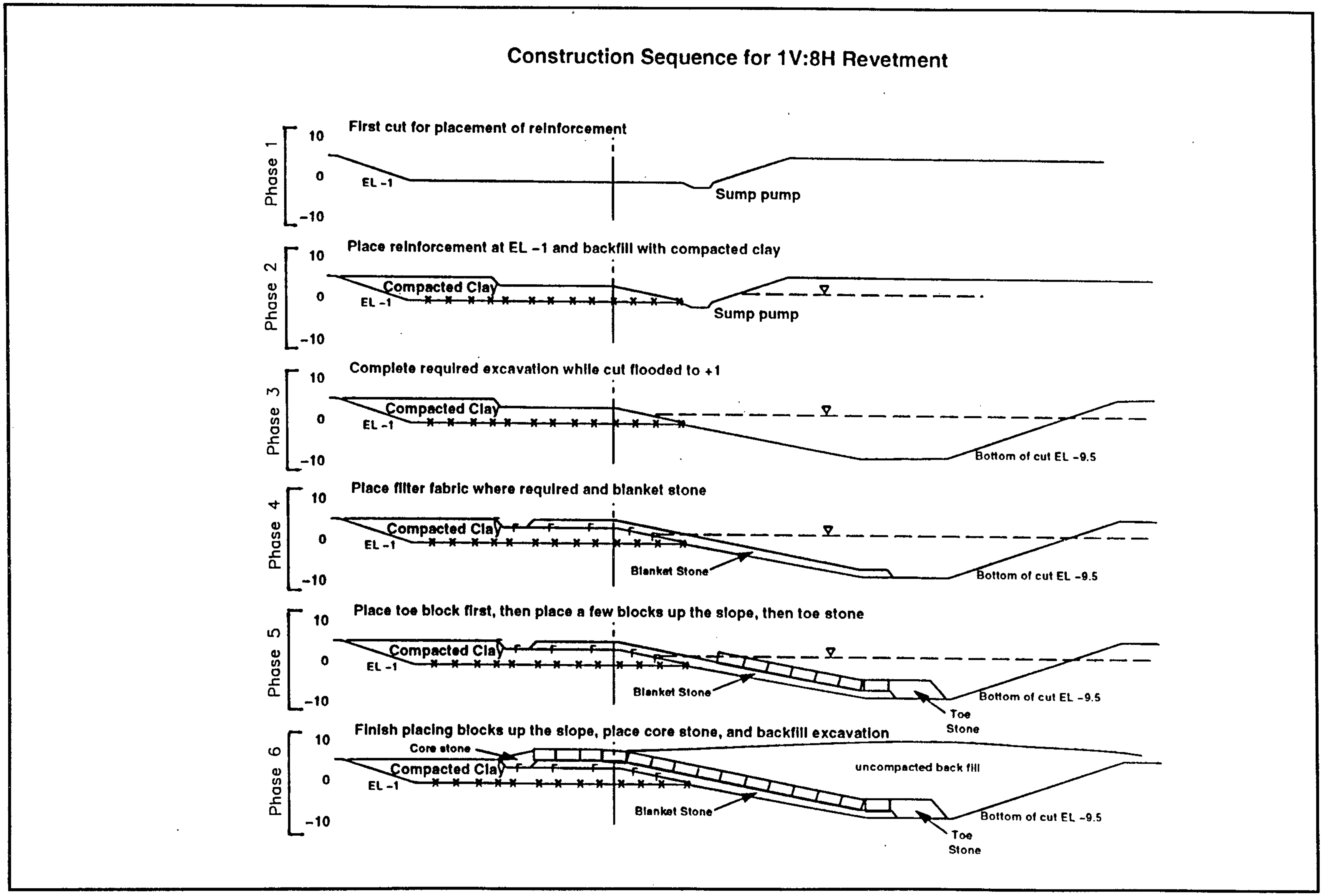


Figure 4. Sequence of construction activities

Mtl	Description	γ pcf	c psf	φ°
1	Fill	120	400	0
2	Med Clay	119	600	0
3	Soft Clay	119	290	0
4	V. Soft Clay	113	130	0
5	Med Clay	108	600	0
6	Stone	130	0	35
7	Block	137	0	0

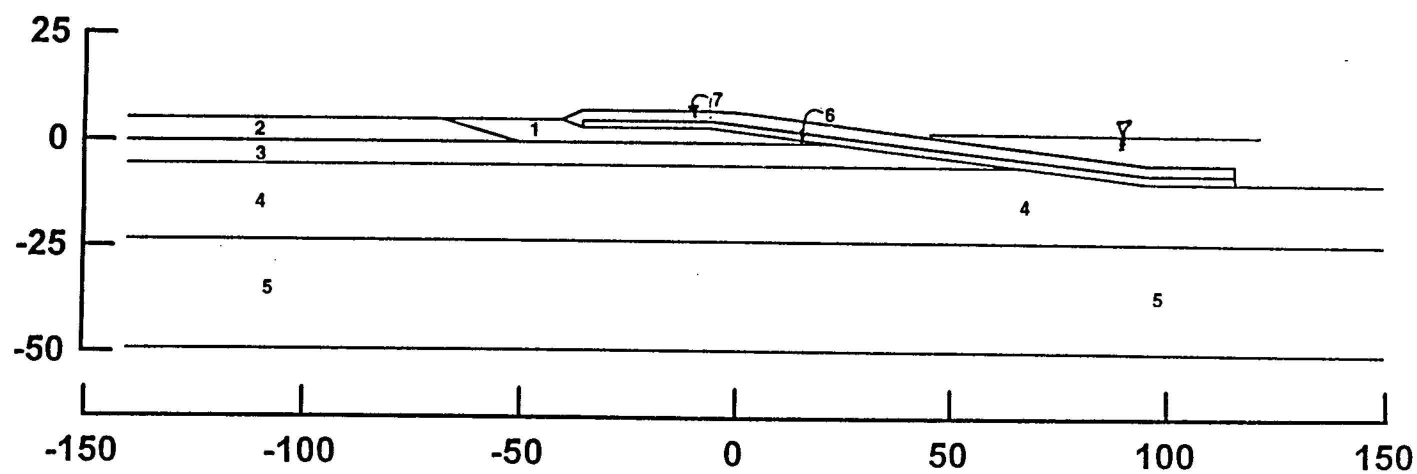


Figure 5. Model of slope used in UTEXAS3 analysis

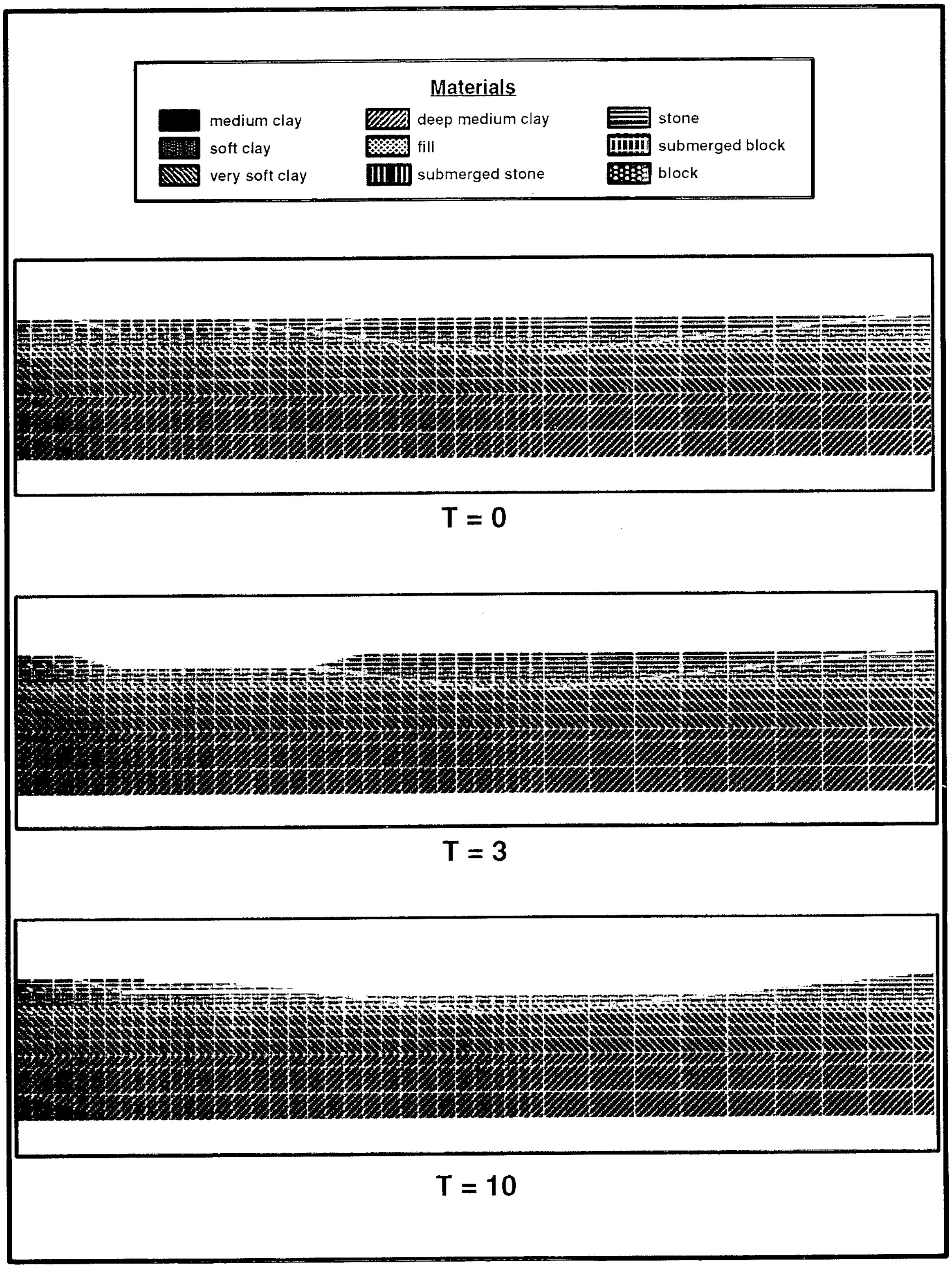


Figure 6. Finite element mesh at construction step numbers 0, 3, and 10

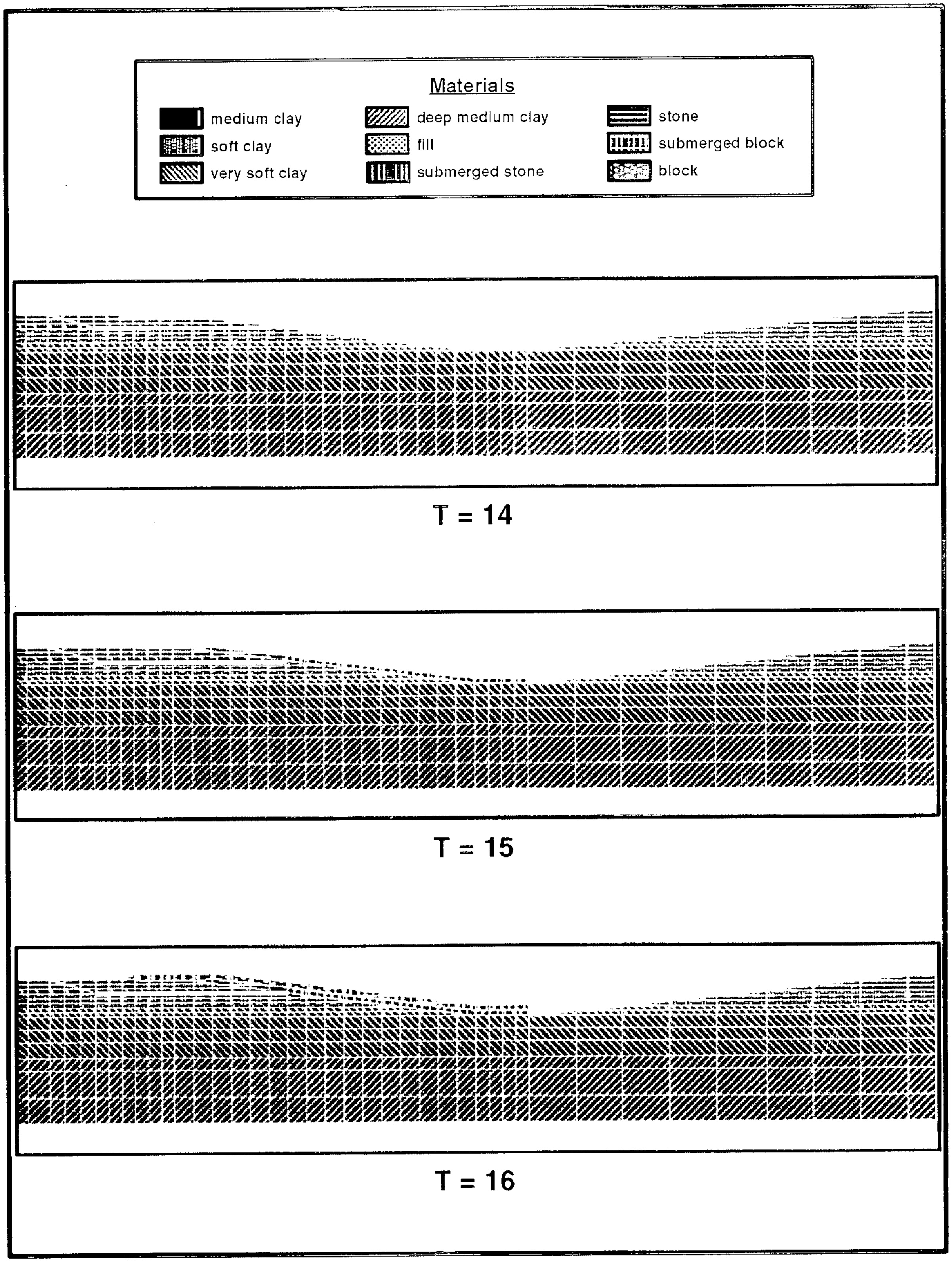


Figure 7. Finite element mesh at construction step numbers 14, 15, and 16

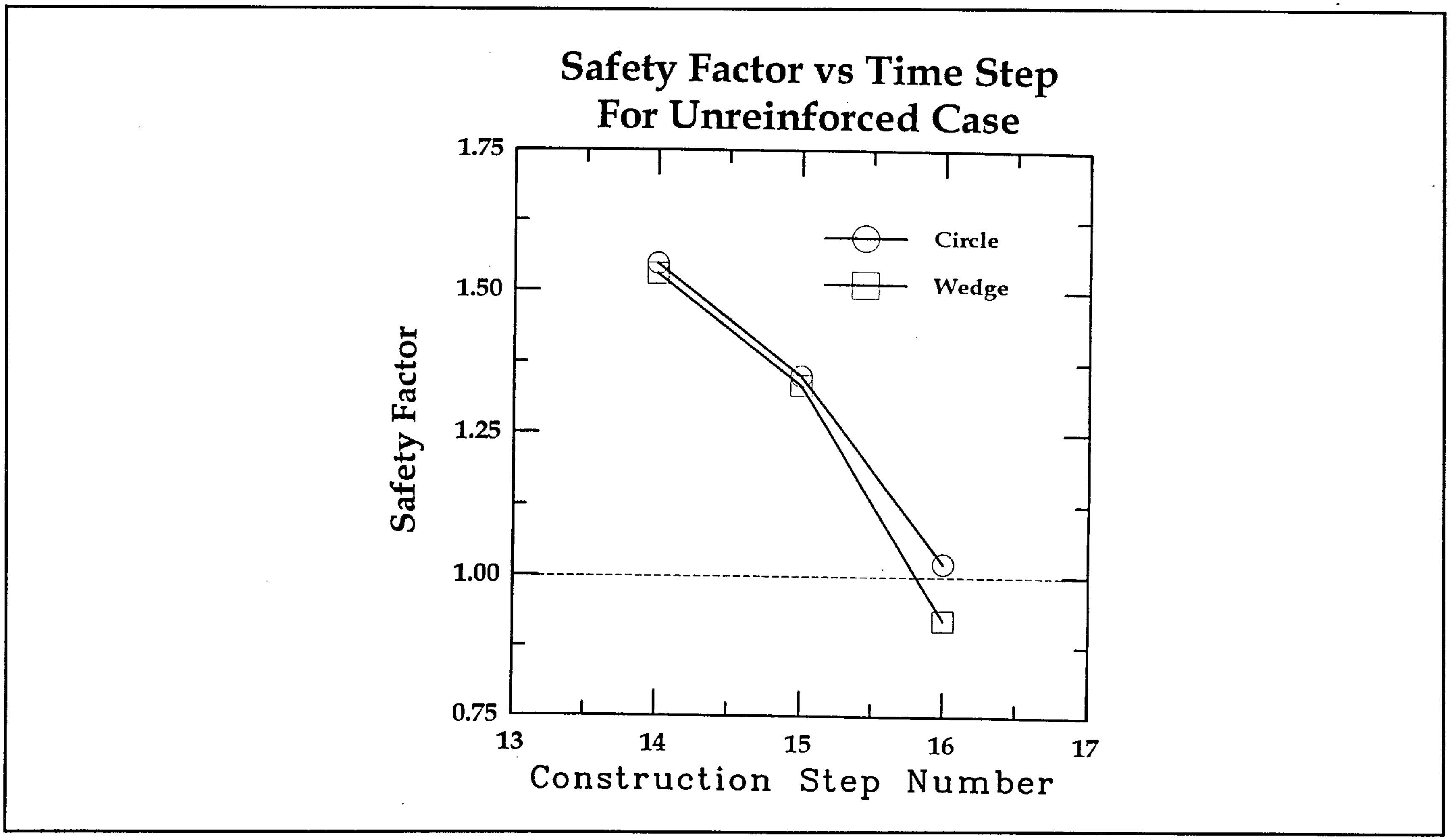
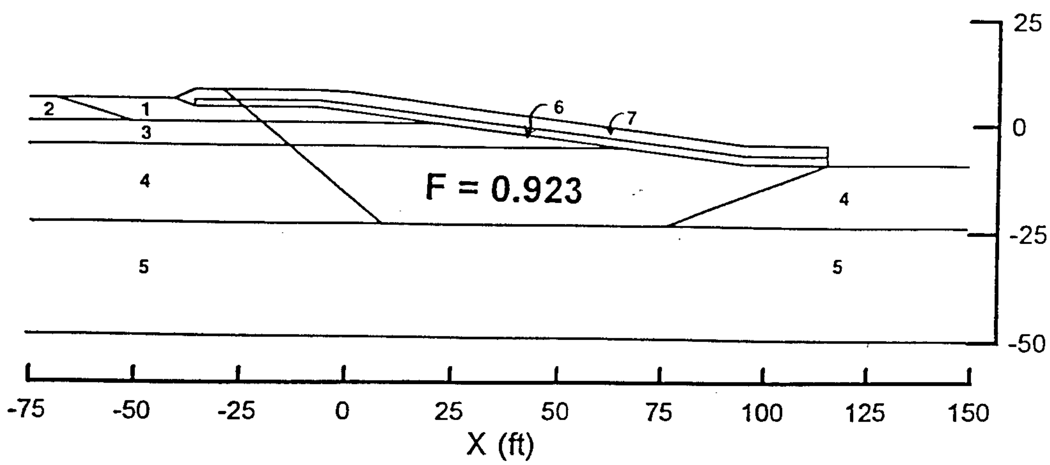


Figure 8. Safety factors for circular and noncircular failure surfaces at the last three construction steps



Mtl	Description	γ pcf	c psf	φ°
1	Fill	120	400	0
2	Med Clay	119	600	0
3	Soft Clay	119	290	0
4	V. Soft Clay	113	130	0
5	Med Clay	108	600	0
6	Stone	130	0	35
7	Block	137	0	0

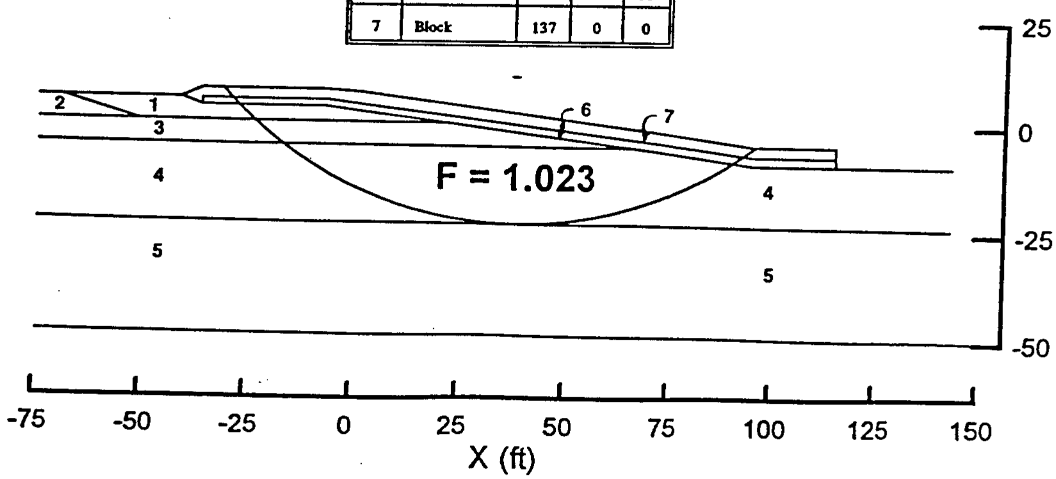


Figure 9. Critical wedge and circular failure surfaces at the end of the last (16th) construction step

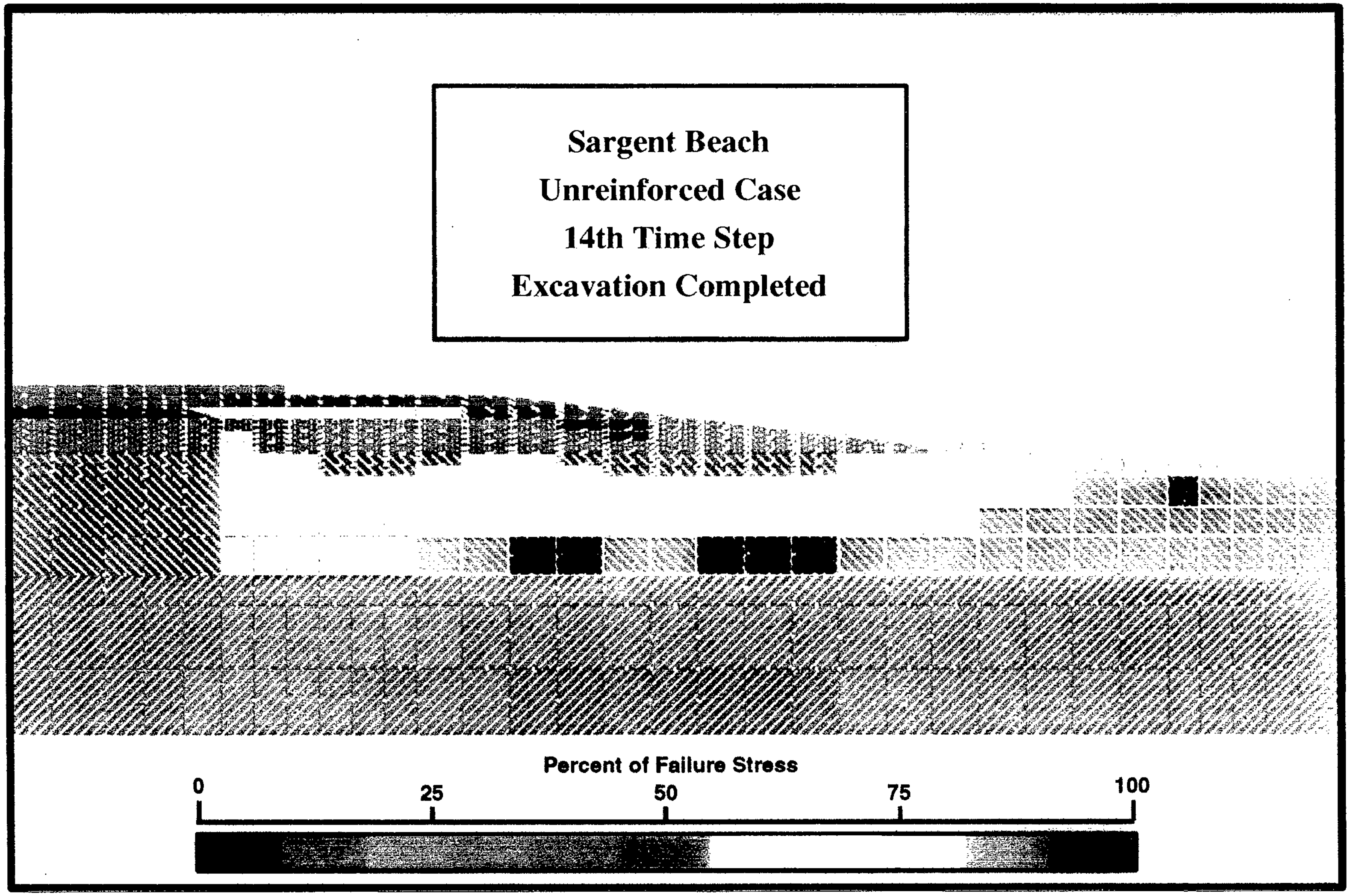


Figure 10. Displacement vectors and mobilized shear stress expressed as a percentage of failure stress at the end of the 14th step when the excavation is completed to elevation -9.5 ft

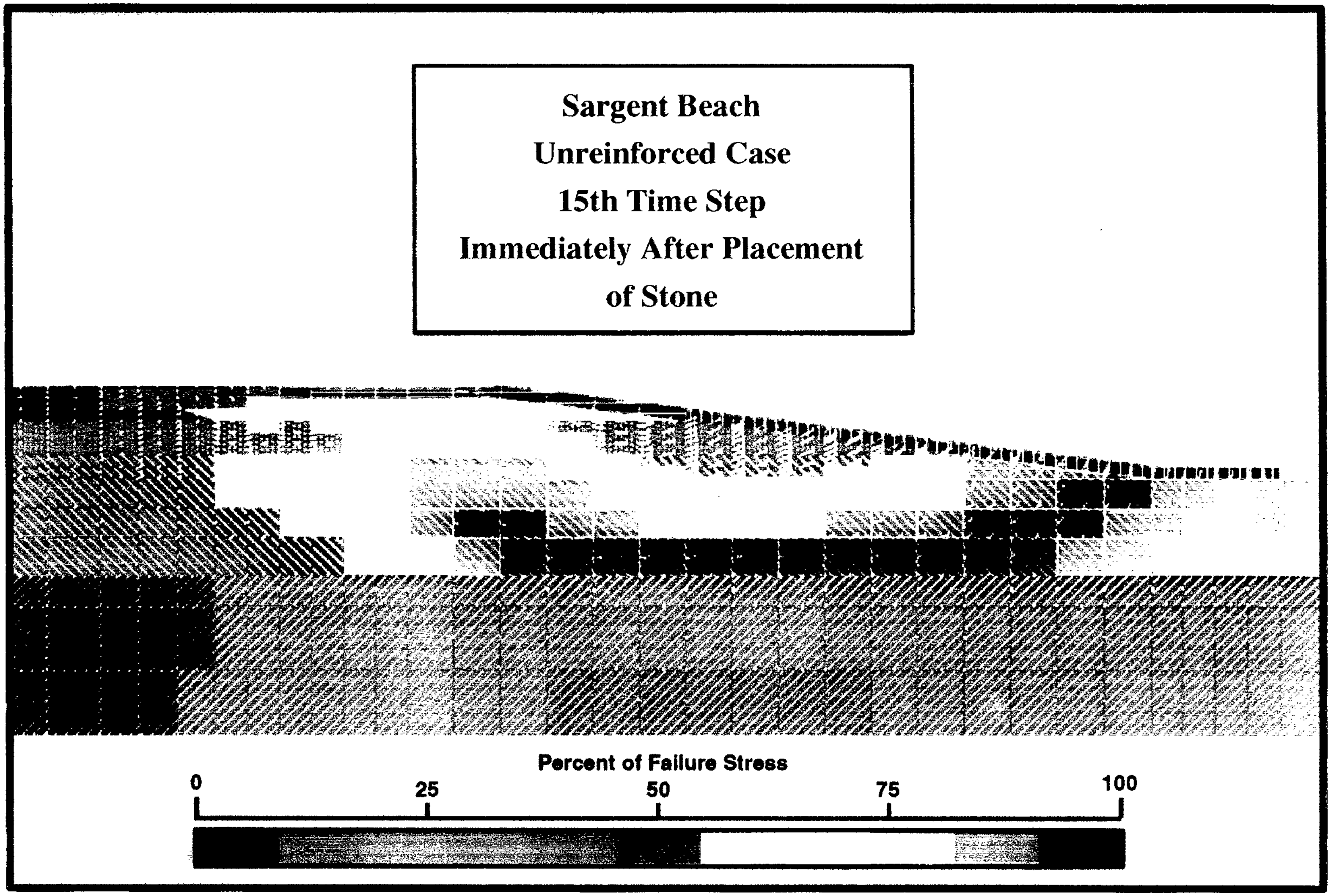


Figure 11. Displacement vectors and mobilized shear stress expressed as a percentage of failure stress at the end of the 15th construction step after the stone blanket is placed

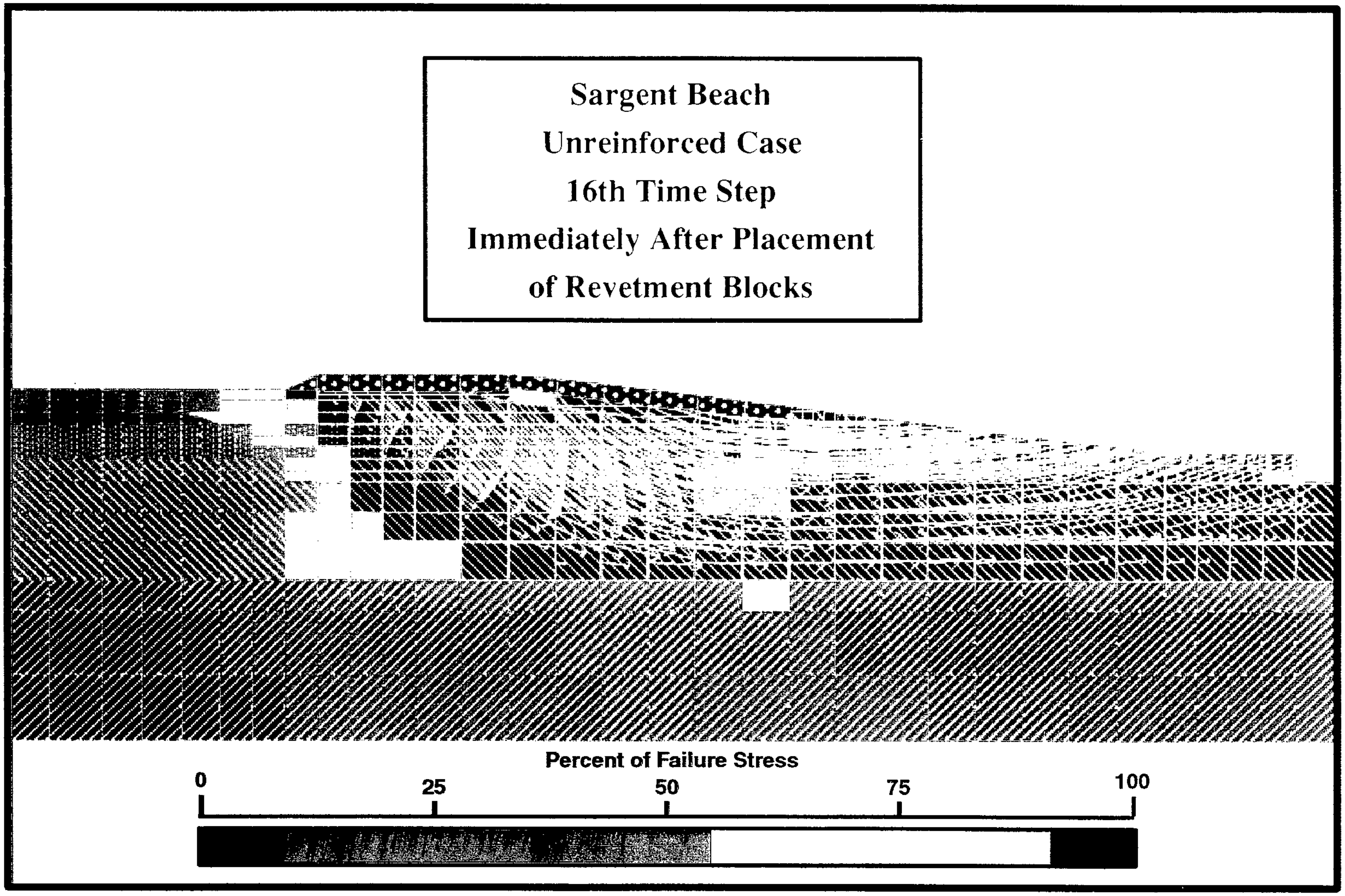


Figure 12. Displacement vectors and mobilized shear stress expressed as a percentage of failure stress at the end of the 16th construction step after the revetment blocks are placed

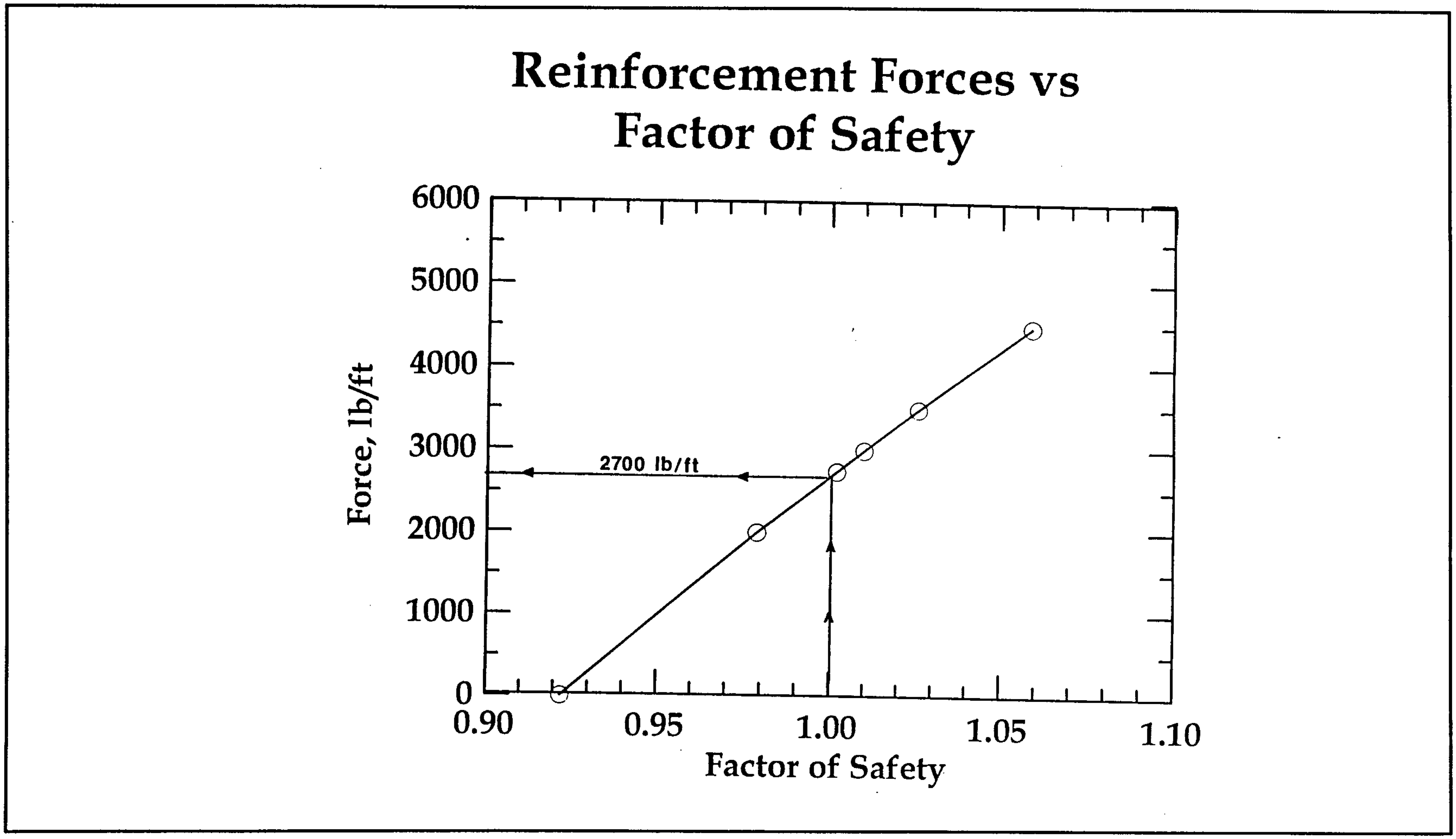


Figure 13. Factor of safety versus force for reinforced revetment slope

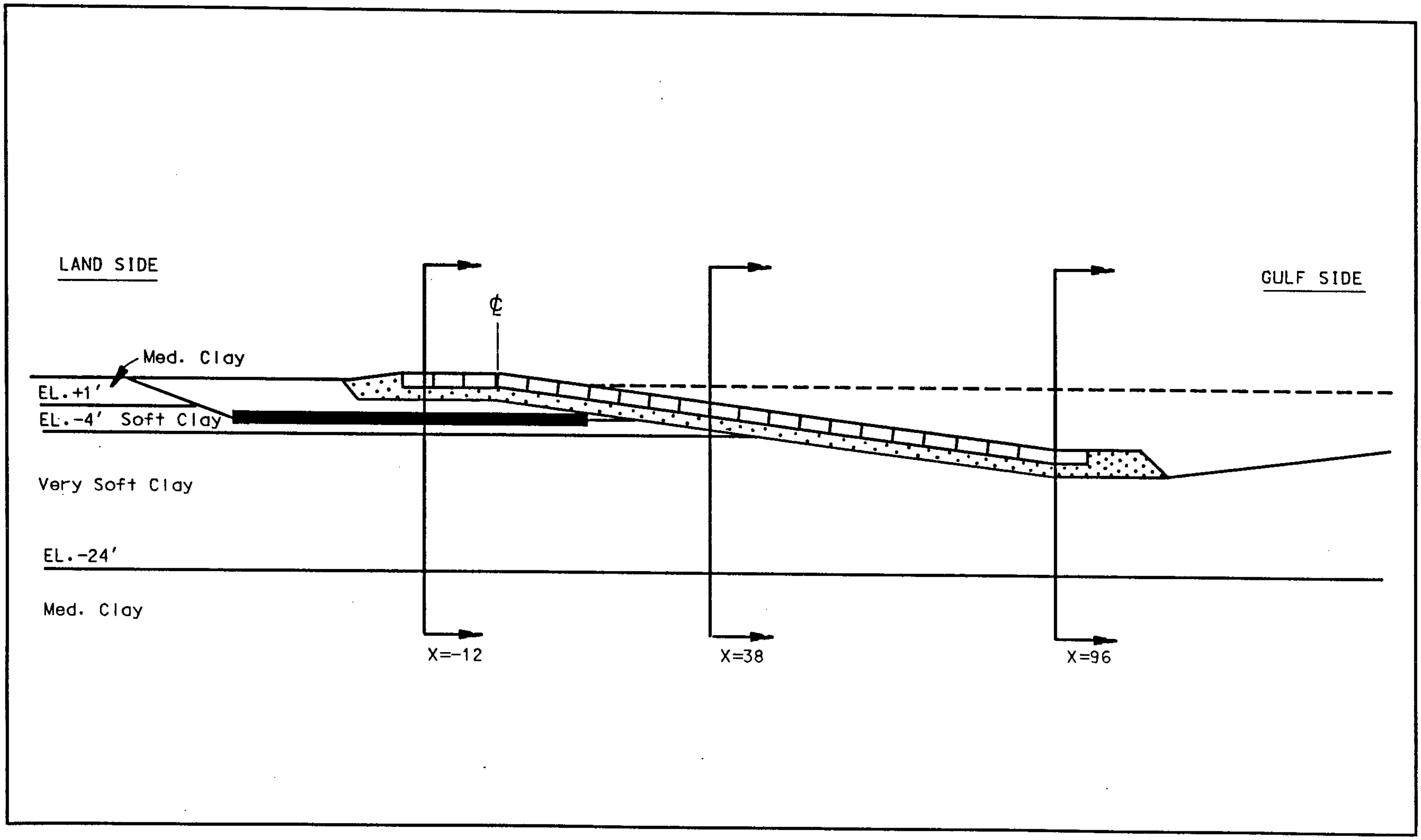


Figure 14. Locations of lateral displacement profiles

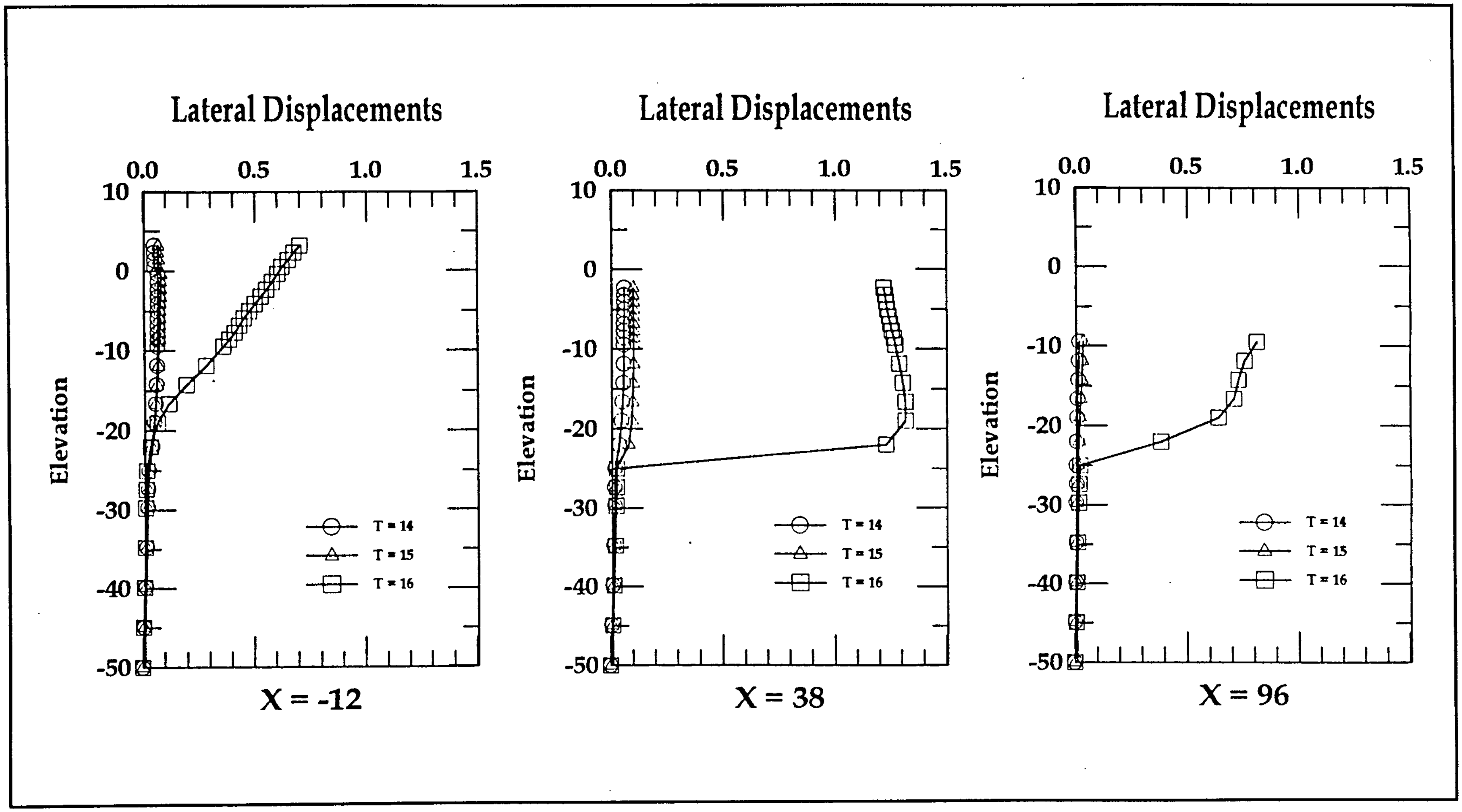


Figure 15. Lateral displacements after construction steps 14, 15, and 16 at top, midslope, and toe locations for reinforcement stiffness of 90,000 lb/ft^2

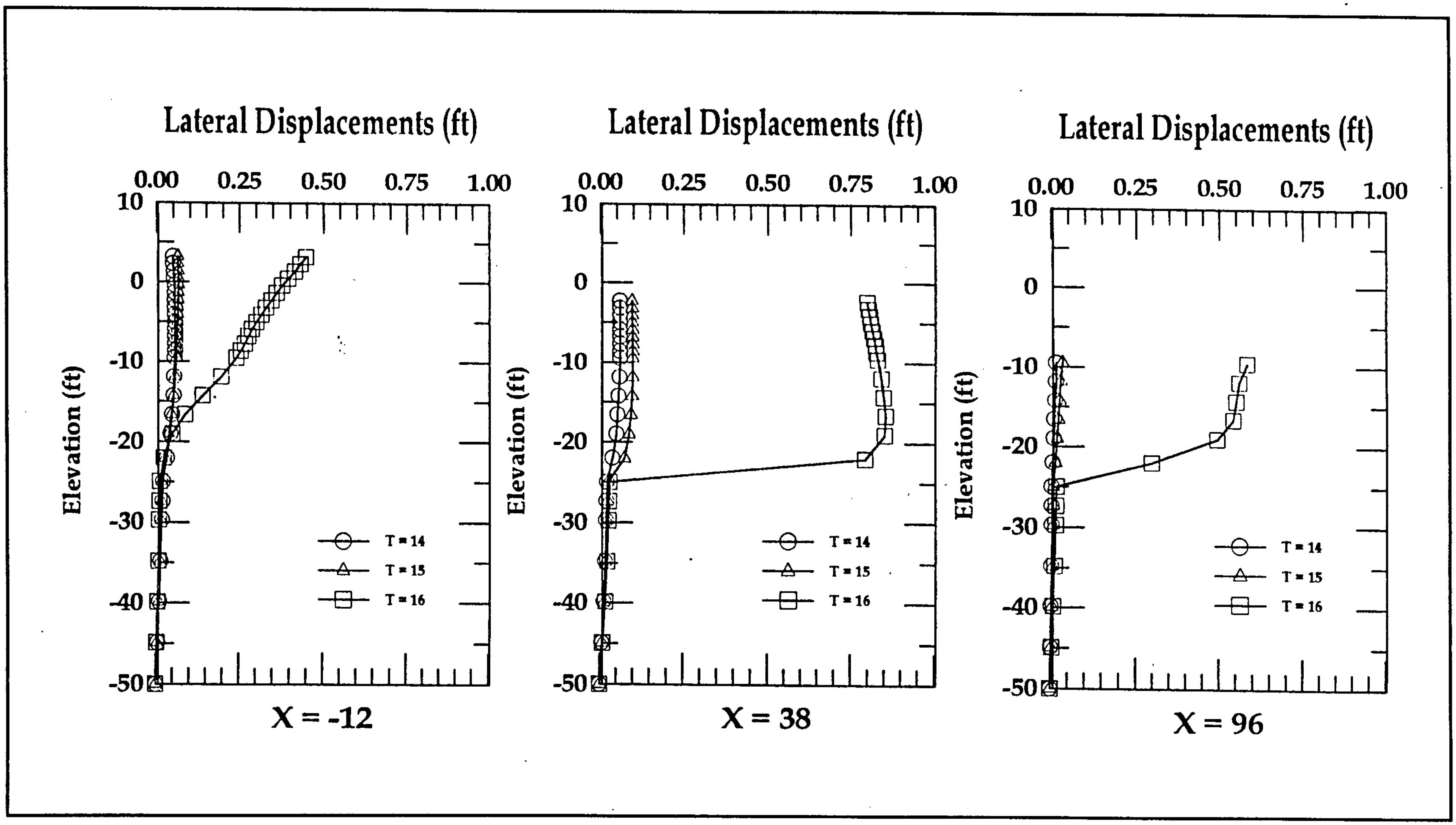


Figure 16. Lateral displacements after construction steps 14, 15, and 16 at top, midslope, and toe locations for reinforcement stiffness of 200,000 lb/ft^2

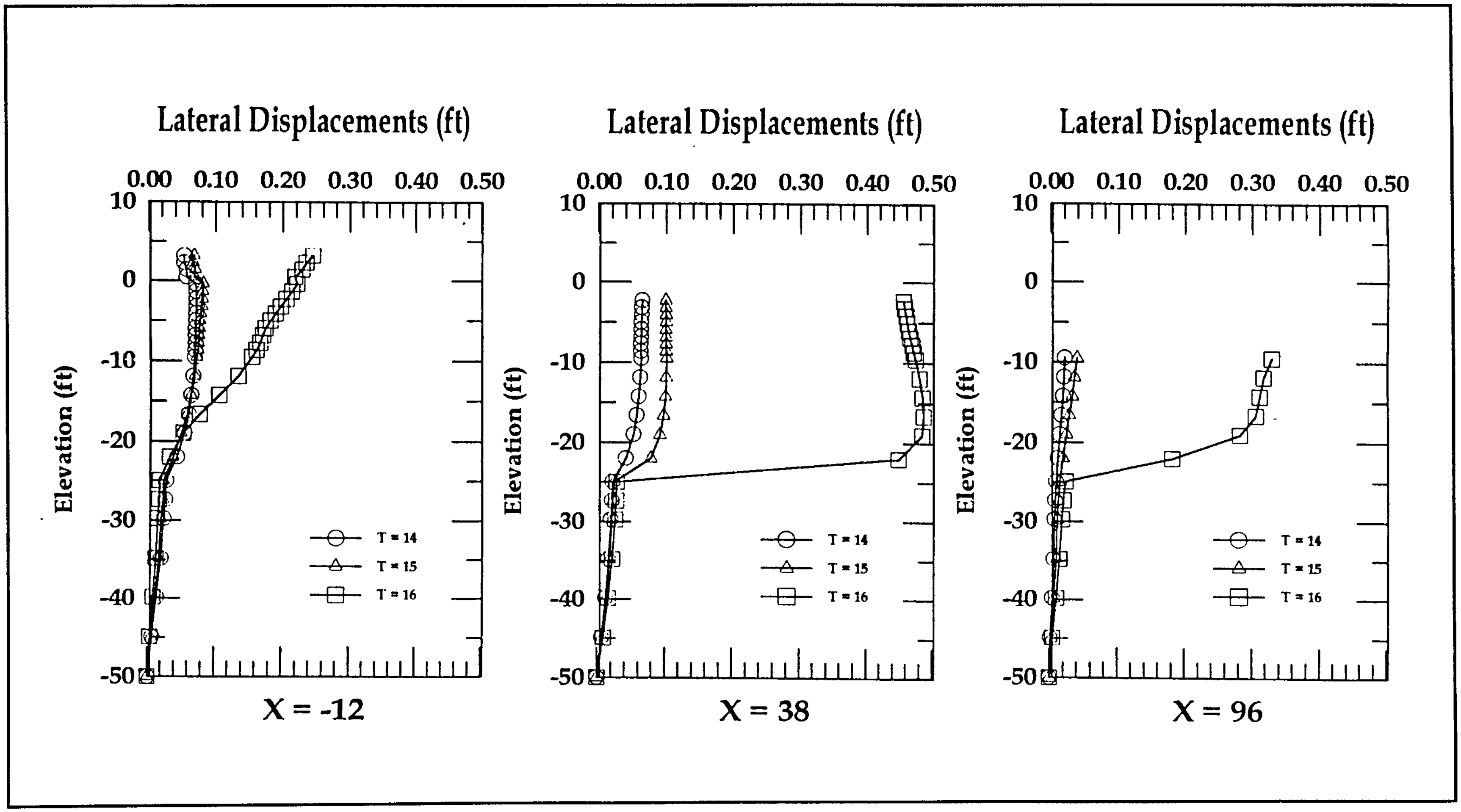


Figure 17. Lateral displacements after construction steps 14, 15, and 16 at top, midslope, and toe locations for reinforcement stiffness of 500,00 lb/ft^2

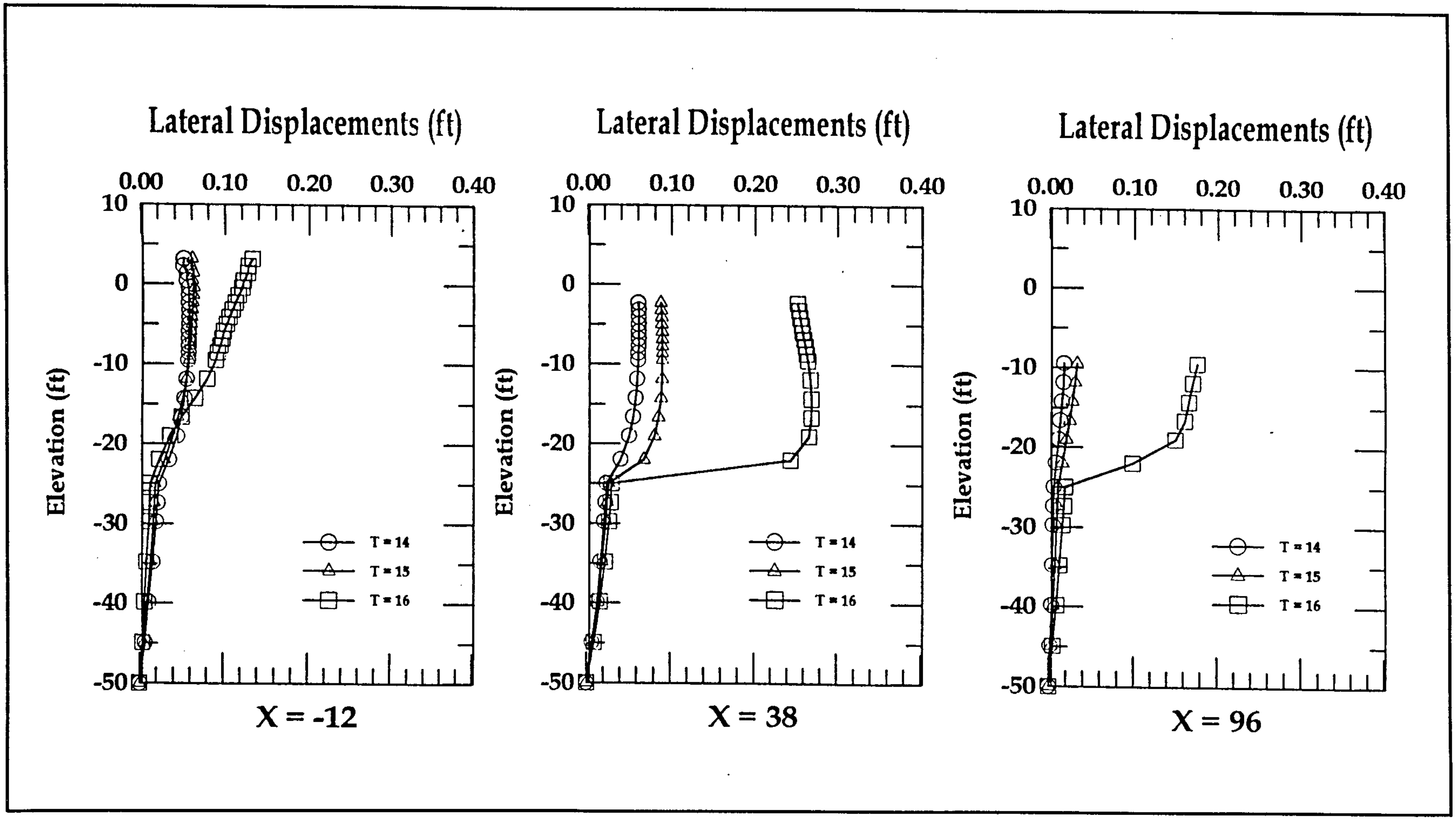


Figure 18. Lateral displacements after construction steps 14, 15, and 16 at top, midslope, and toe locations for reinforcement stiffness of 1,500,000 lb/ft^2

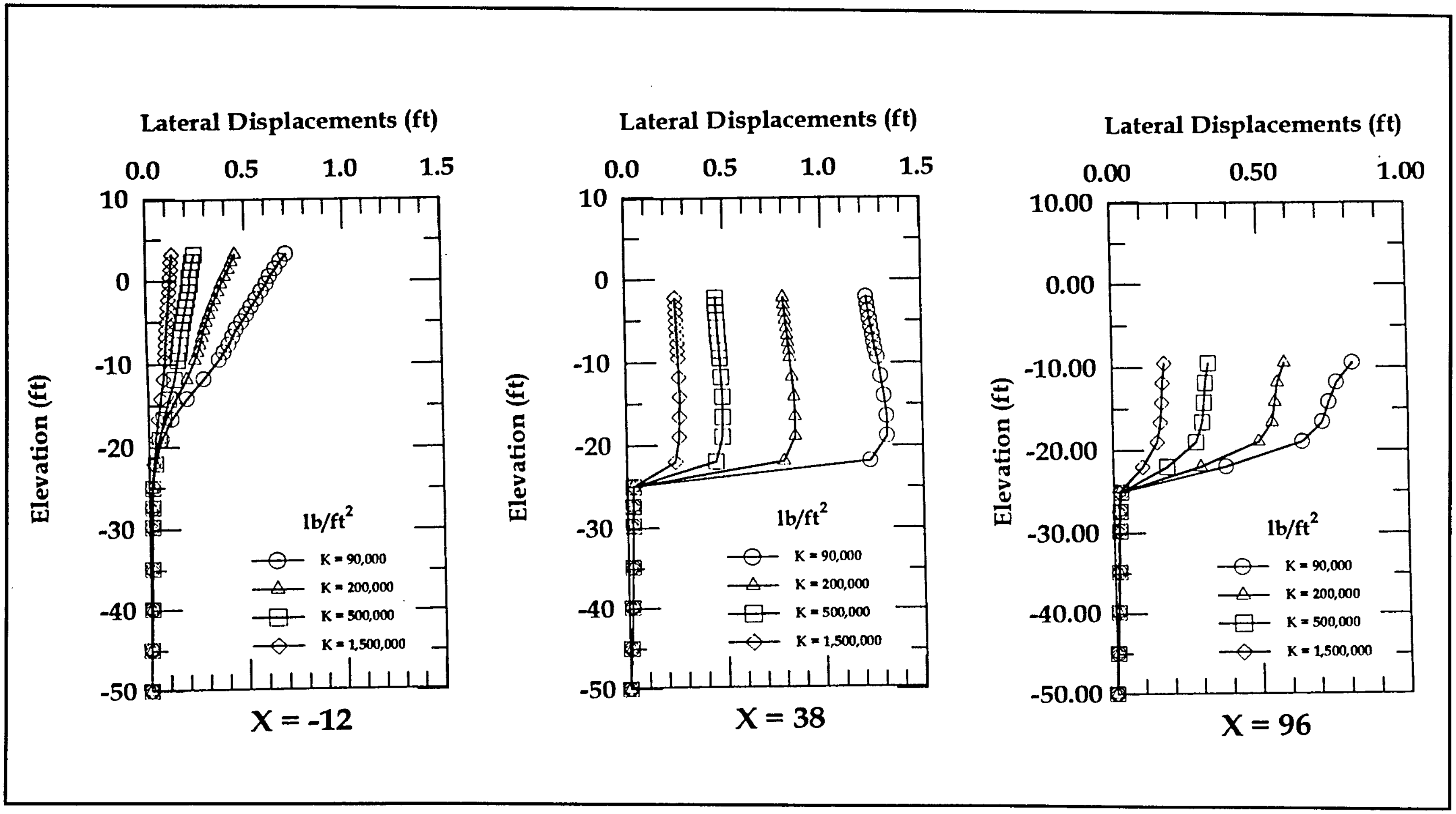


Figure 19. Comparison of lateral displacement profiles for reinforcement stiffness of 90,000, 200,000, 500,000, and 1,500,000 lb/ft^2 at the top, midslope, and toe locations after the sixteenth construction step

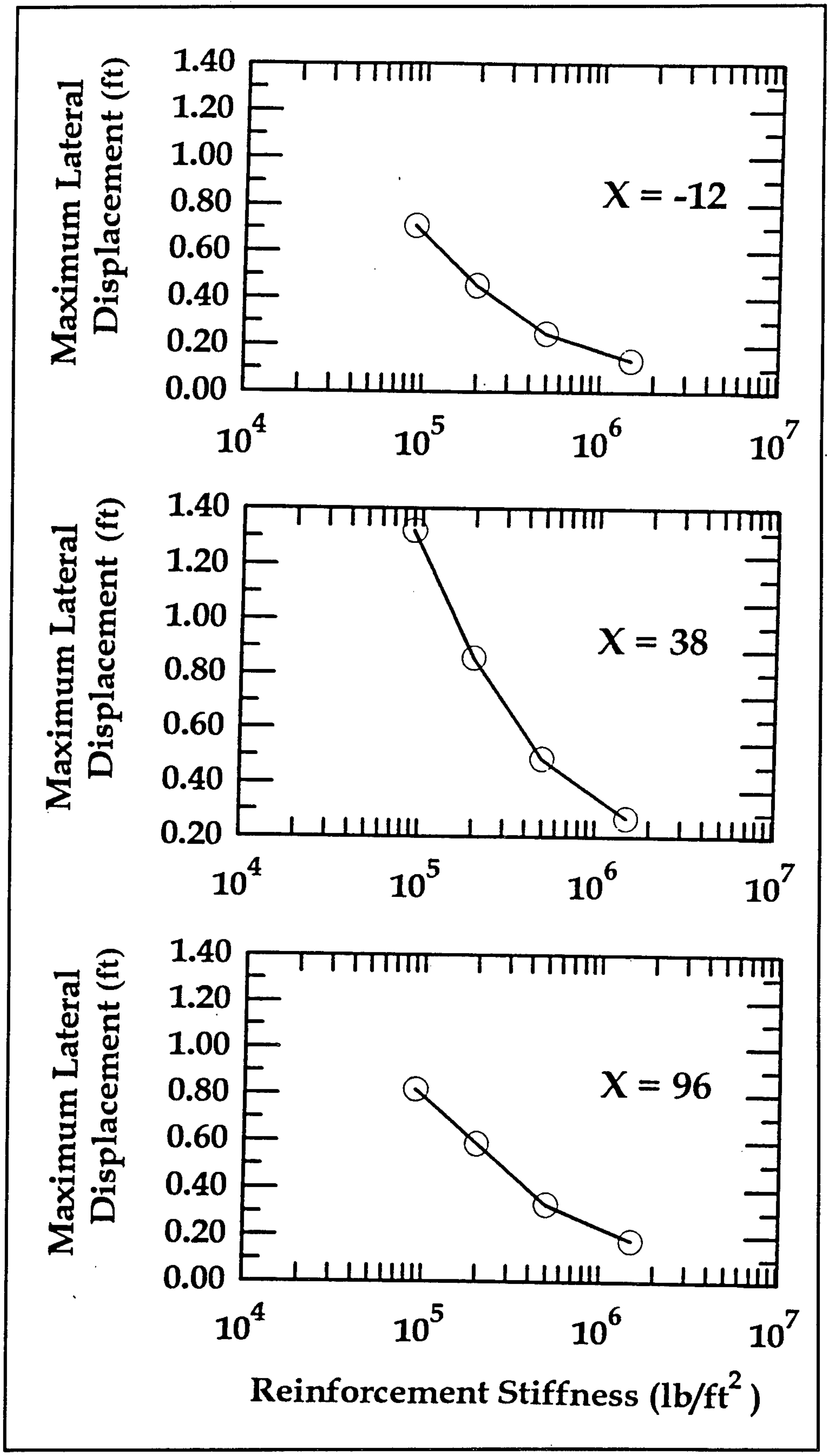


Figure 20. Peak lateral displacements in the midslope profile (X = 38 ft) versus reinforcement stiffness

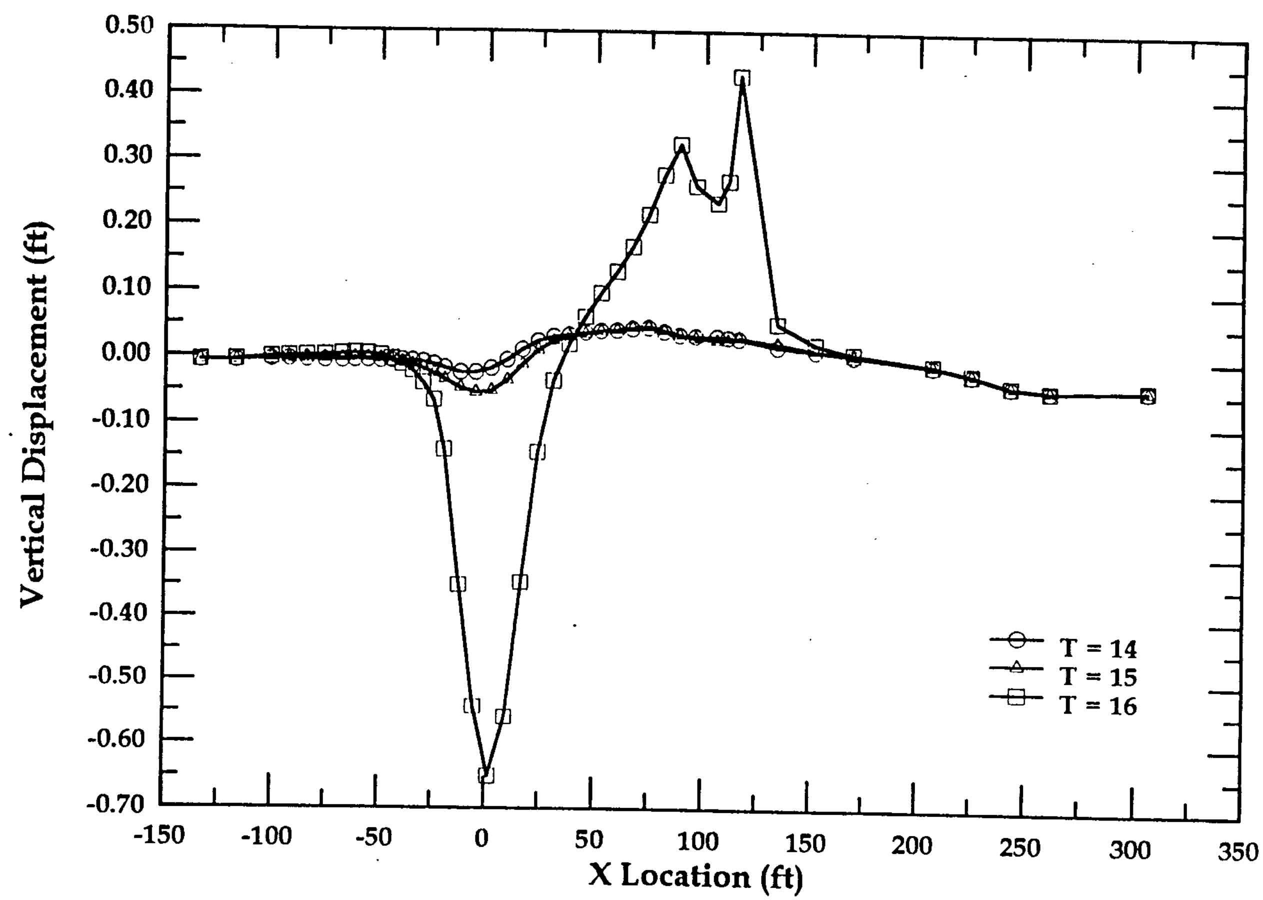


Figure 21. Vertical displacements at elevation -9.5 ft for construction step numbers 14, 15, and 16 for a reinforcement stiffness of 90,000 lb/ft

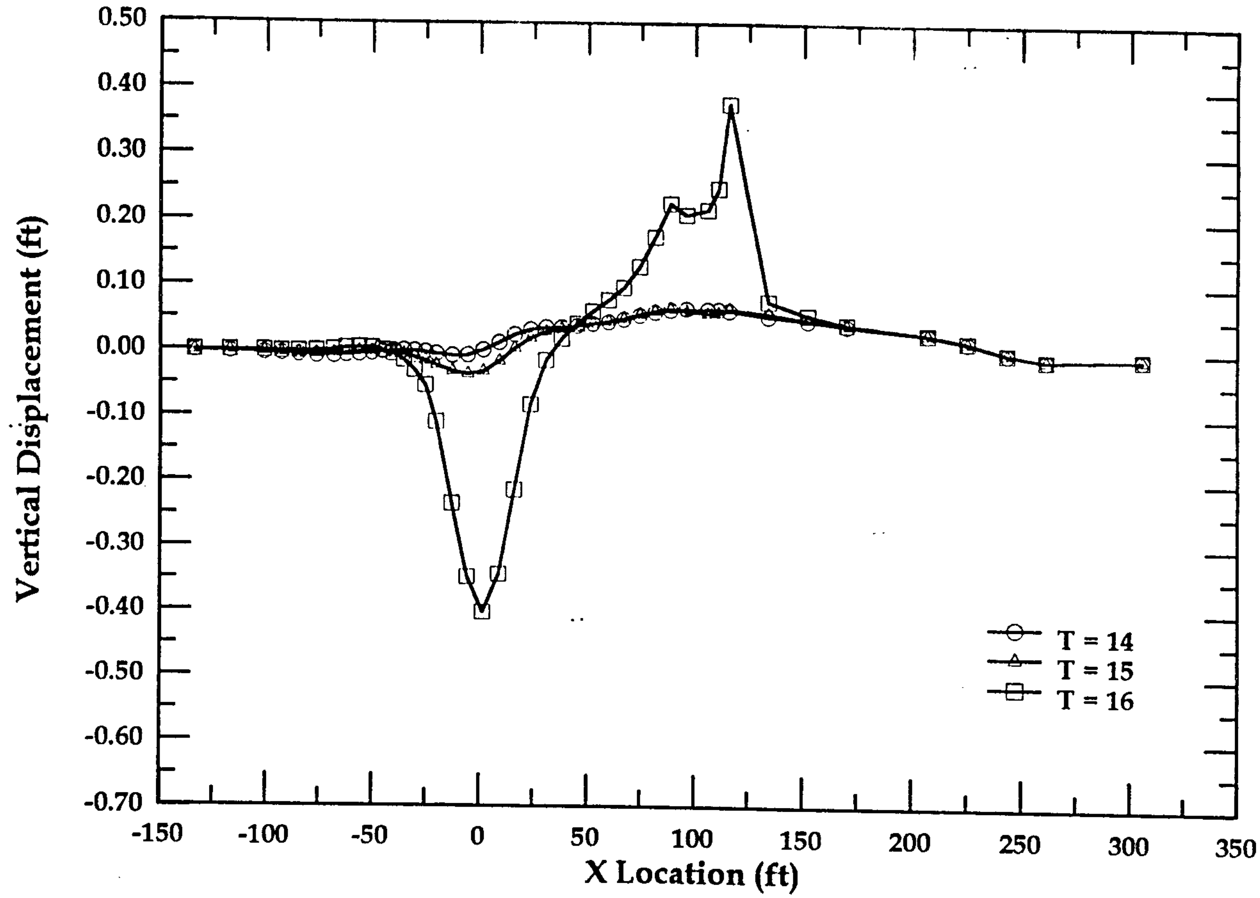


Figure 22. Vertical displacements at elevation -9.5 ft for construction step numbers 14, 15, and 16 for a reinforcement stiffness of 200,000 lb/ft

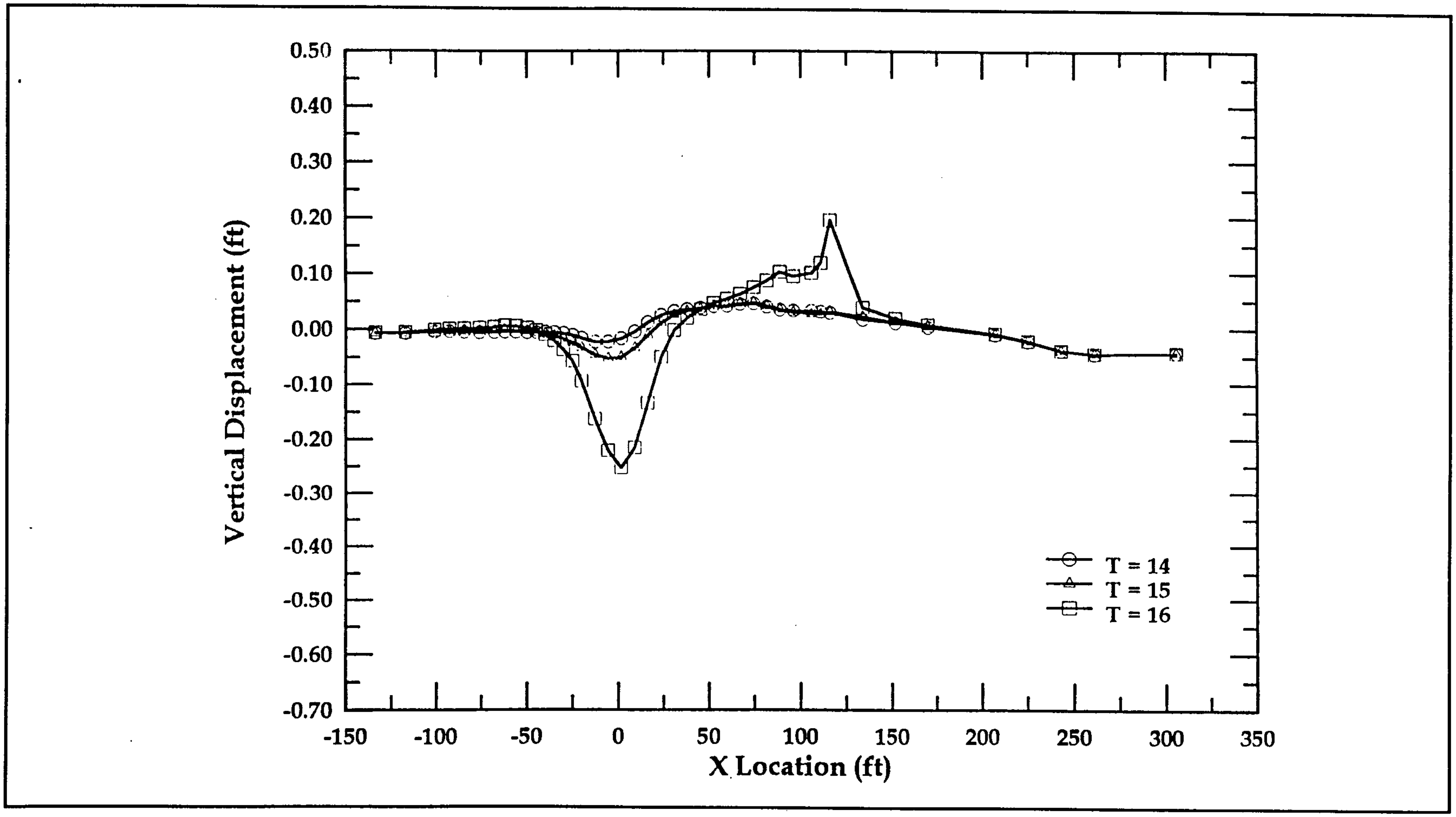


Figure 23. Vertical displacements at elevation -9.5 ft for construction step numbers 14, 15, and 16 for a reinforcement stiffness of 500,000 lb/ft

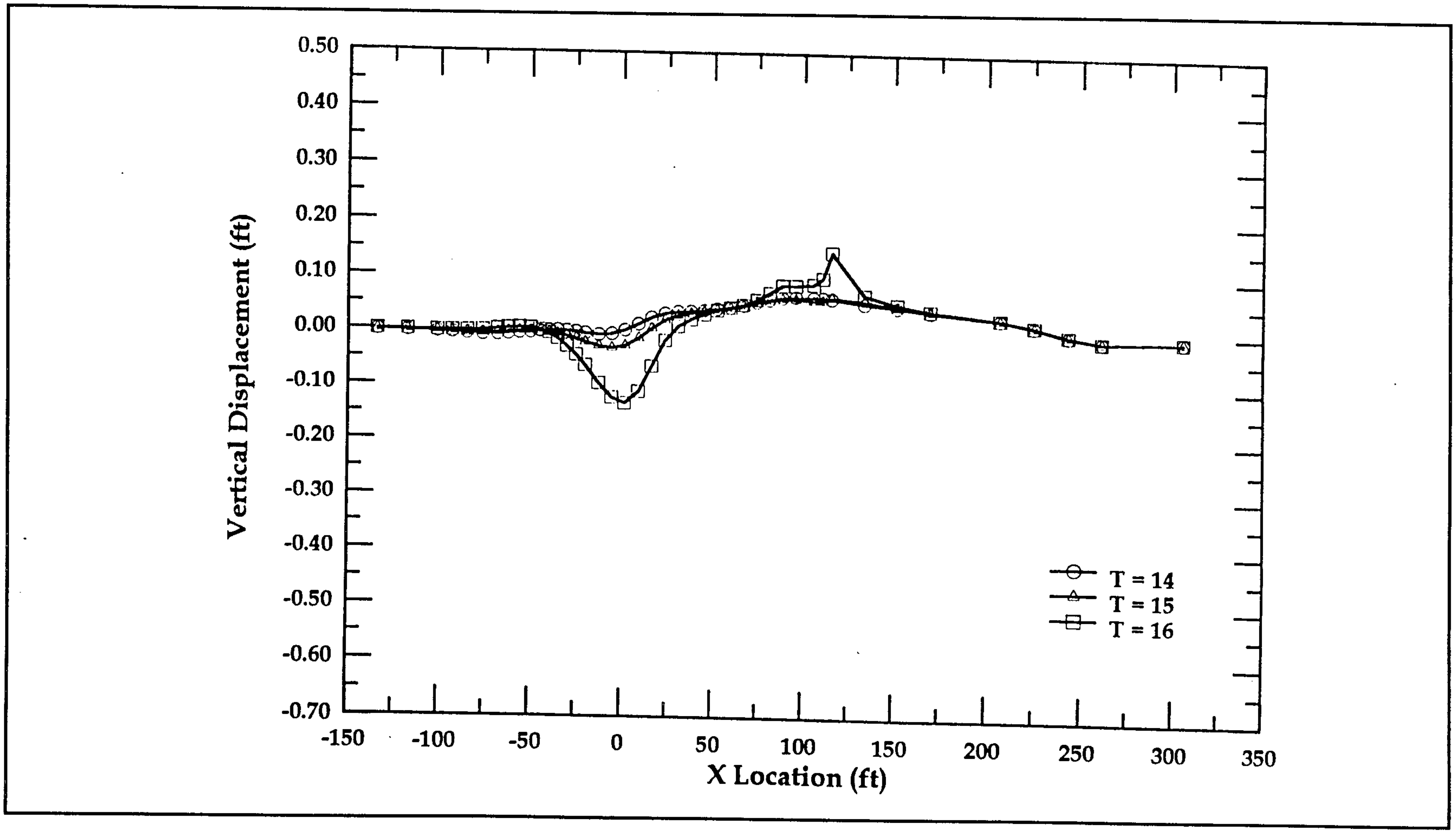


Figure 24. Vertical displacements at elevation -9.5 ft for construction step numbers 14, 15, and 16 for a reinforcement stiffness of 1,5000,000 lb/ft

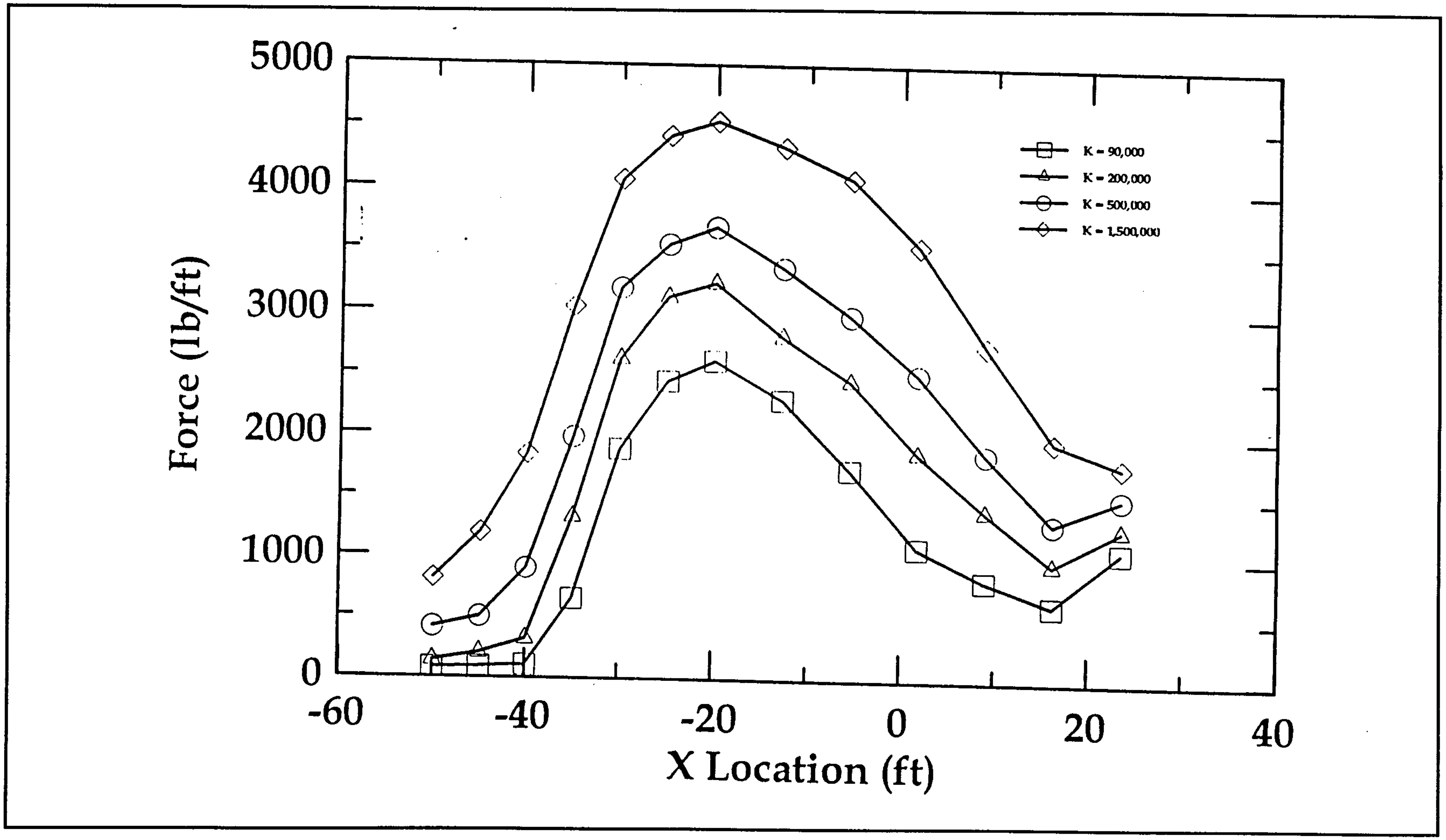


Figure 25. Force distributions in reinforcement for different stiffnesses at construction step 16

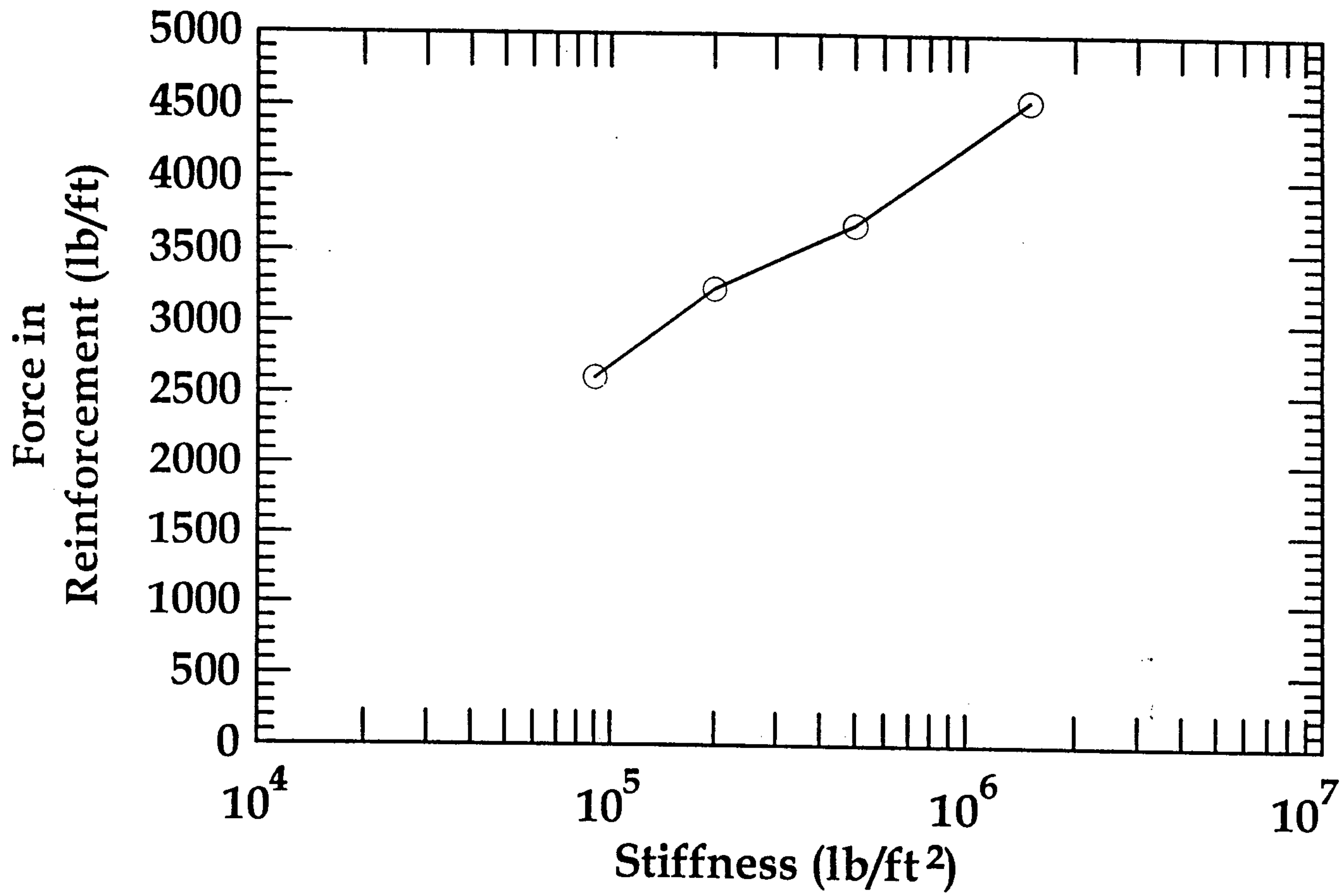


Figure 26. Peak force in reinforcement versus stiffness

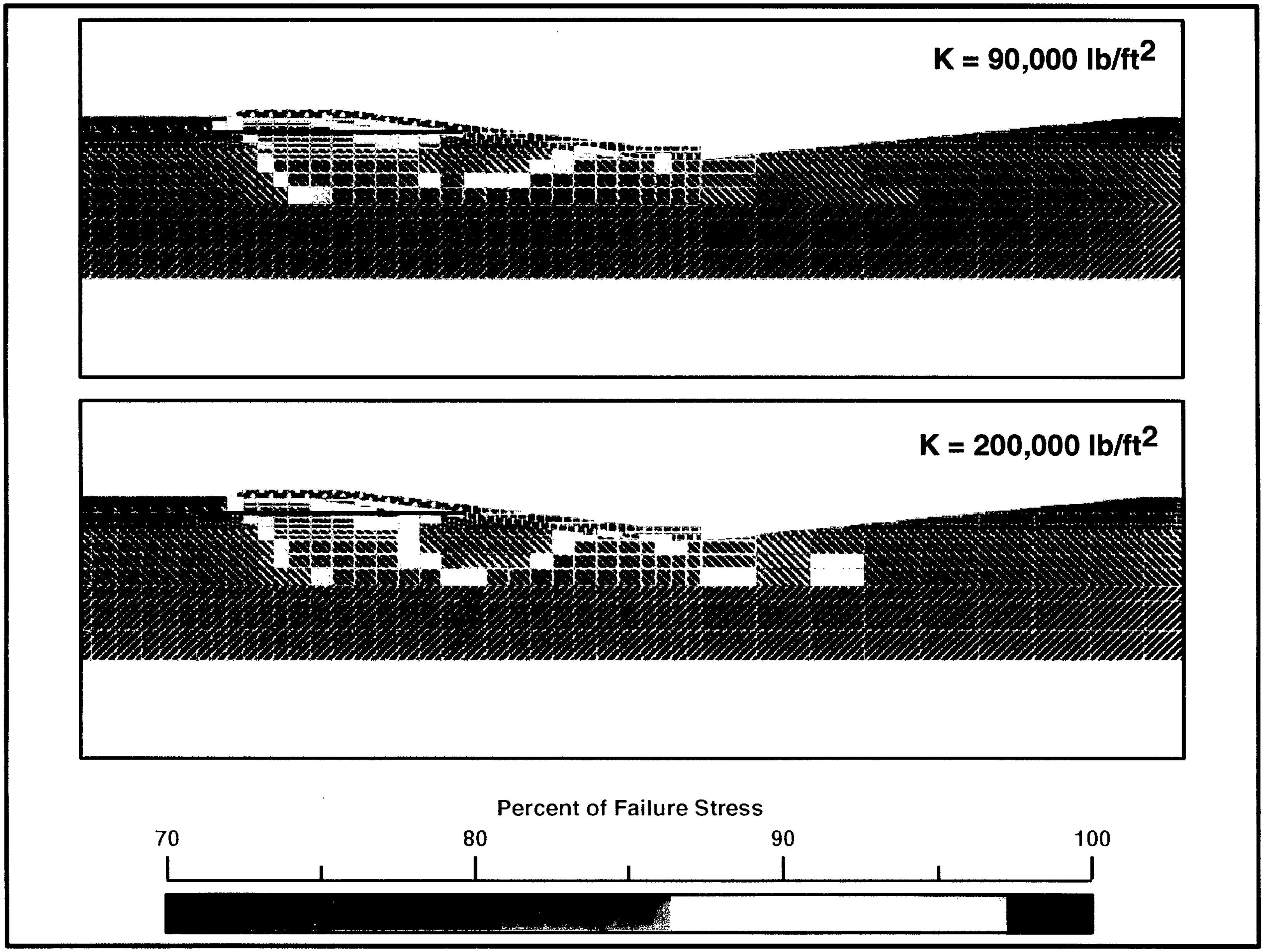


Figure 27. Percentage of failure stress for stiffnesses of 90,000 and 200,000 lb/ft^2

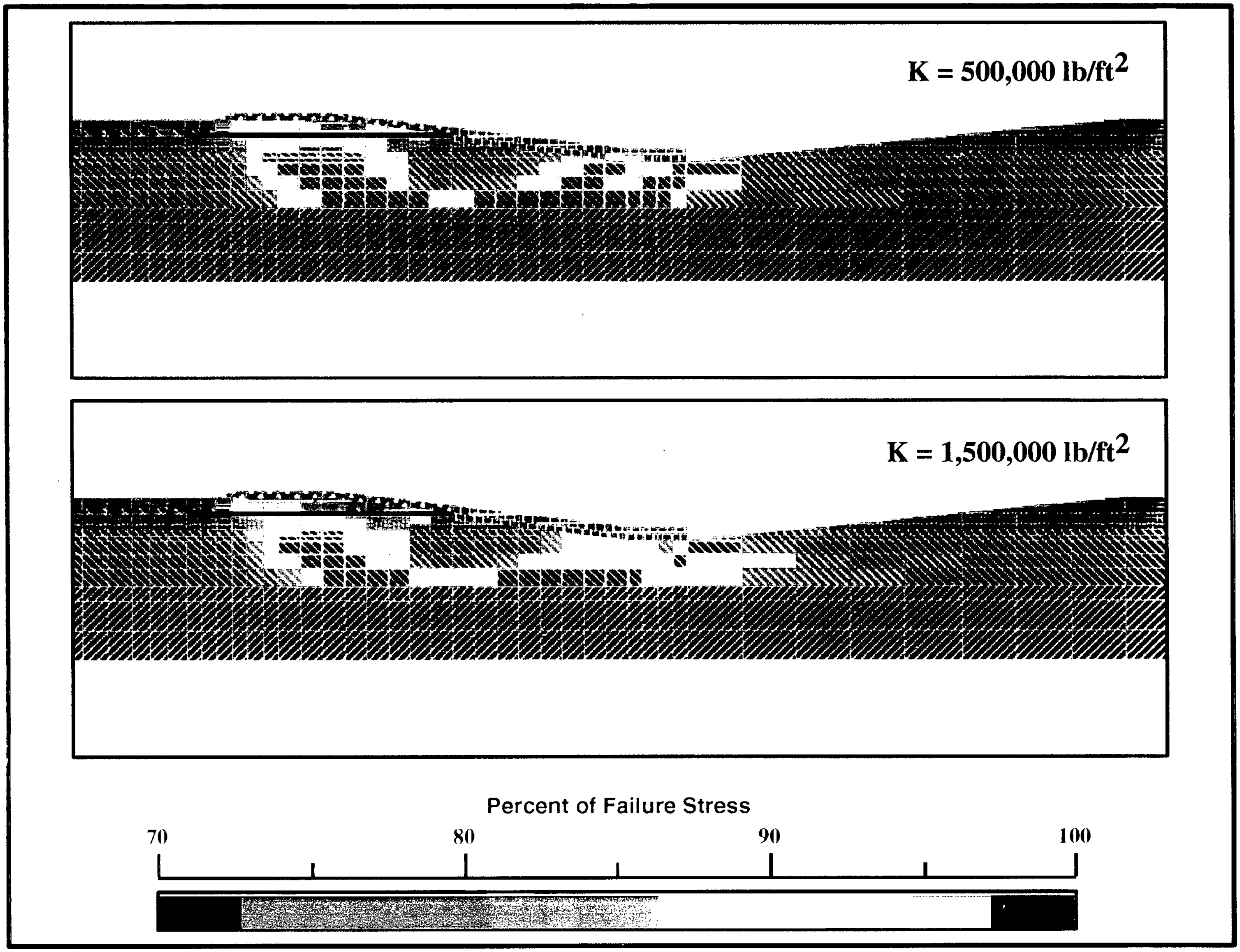


Figure 28. Percentage of failure stress for stiffnesses of 500,000 and 1,500,000 lb/ft^2

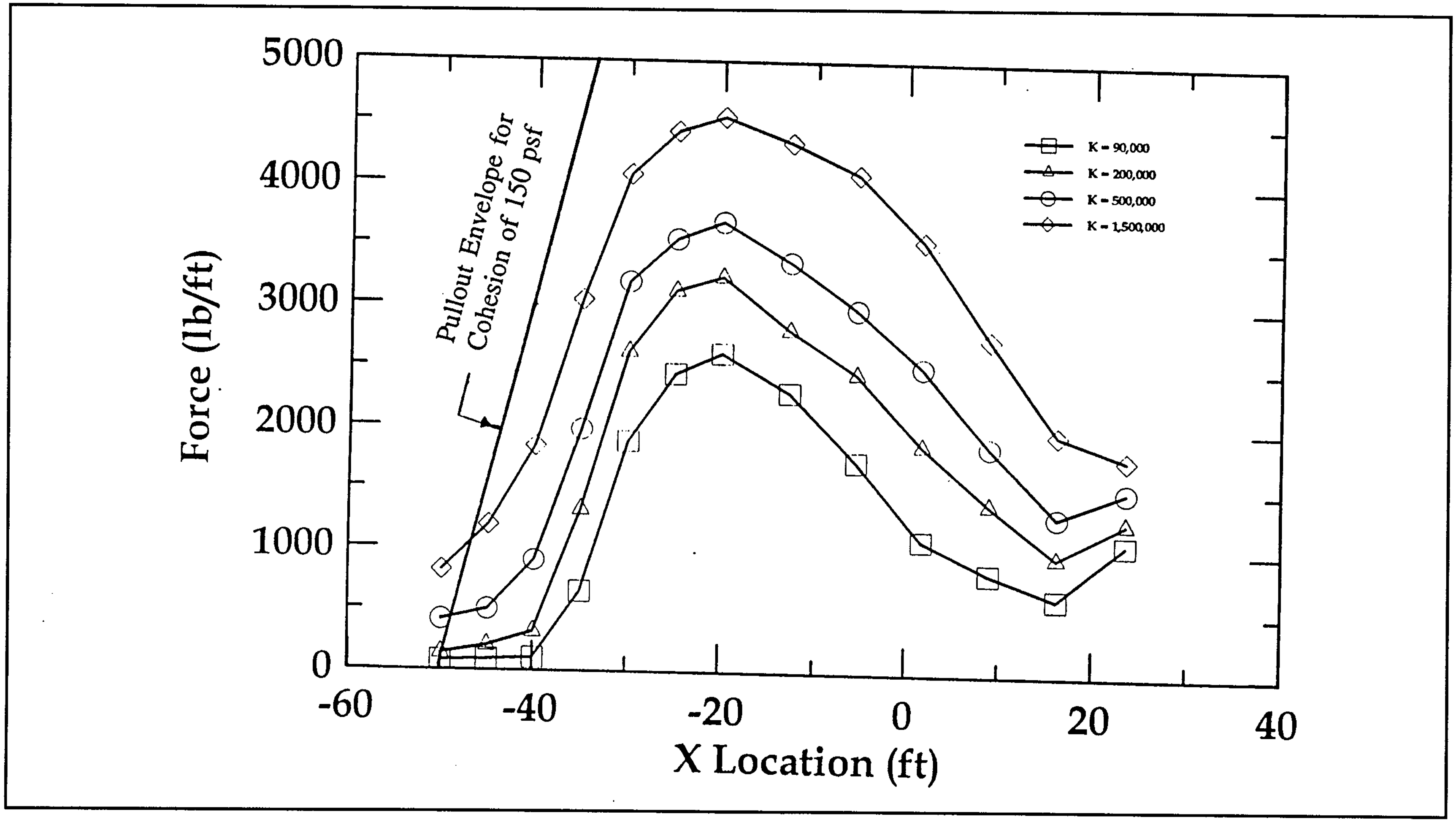


Figure 29. Comparison of pullout forces with force distributions in the geotextile

Figure 30. Aerial view of Sargent Beach Erosion Protection Project

Figure 31. Placement of blanket stone and granite blocks

Table 1
Finite Element Steps

Phase Number	Step Number	Description
Initial Conditions	0	Initial state represented by K_o level ground conditions
1	1, 2, 3	Initial excavation to Elevation -0.4 ft, the elevation at which the reinforcement will be placed
2	4	Place reinforcement and first of three lifts of fill over reinforcement
3	5, 6	Place second and third lifts of fill over reinforcement to Elevation +5.0 ft
4	7, 8, 9, 10, 11, 12, 13, 14	Excavate slope and sideslopes of 8:1 (land side) and 10:1 (gulfside) to Elevation -9.5 ft over eight construction steps
5	15	Place blanket stone on 8:1 slope
6	16	Place revetment block over blanket stone

Table 2
Descriptions of Parameters Used in the Finite Element Analysis

Parameter	Description
S_u	Undrained shear strength
γ	Unit Weight
E	Young's modulus
μ	Poisson's ratio
K_o	Coefficient of earth pressure at rest
F_s	Endochronic parameter related to shear strength
ϕ'	Friction angle

Table 3
Material Parameters and Properties for Finite Element Analysis

Mtl No.	Description	Su psf	γ pcf	E psf	μ	K_o	F_s psf	ϕ' degrees
1	Med Clay	600	119.0	600,000	0.45	0.999	804.6	----
2	Soft Clay	290	119.0	290,000	0.45	0.999	388.6	----
3	Very Soft Clay	130	113.0	130,000	0.45	0.999	174.2	----
4	Med Clay	600	108.0	600,000	0.45	0.999	804.0	----
5	Fill	400	120.0	400,000	0.45	0.999	536.0	----
6	Stone (submerged)	165	67.6	165,000	0.45	0.999	221.1	30
7	Stone (dry)	220	130.0	220,000	0.45	0.999	294.8	30
8	Block (Submerged)	----	87.6	10×10^6	0.30	0.999	----	----
9	Block (dry)	----	150.0	10×10^6	0.30	0.999	----	----

Table 4
Results of UTEXAS3 Analysis Reinforcement Force Versus Factor of Safety

Force (lb/ft)	Factor of Safety
0	0
2,000	0.979
2,750	1.002
3,000	1.010
3,500	1.026
4,500	1.059

Table 5
Peak Reinforcement Forces for Different Stiffnesses from Finite Element Analysis

Stiffness (lb/ft)	Force (lb/ft)
90,000	2,610
200,000	3,227
500,000	3,685
1,500,000	4,545

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13. ABSTRACT *(Maximum 200 words)*

This report documents finite element and stability analyses performed at the U.S. Army Engineer Waterways Experiment Station for the U.S. Army Engineer District, Galveston. These analyses aided in the design of a reinforced slope faced with concrete revetment blocks for the Sargent Beach Erosion Protection Project of the Gulf Intracoastal Waterway. The finite element method was used to simulate the construction process and predict the behavior at different stages of the construction process. Excavation, filling, and placement of geosynthetic reinforcement were the construction processes modeled. The principle objectives of the study were to use the results of the analyses to gain insight toward evaluating whether or not the predicted behavior of the slope would meet the stability requirements of the project. The direction and magnitude of foundation movements were estimated from the finite element calculations. The effect of reinforcement stiffness on the predicted performance of the slope was also evaluated and used by district designers in the selection of an appropriate reinforcing material.

14. SUBJECT TERMS

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Finite element method	Slope stability

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